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GOLDEN GATE BRIDGE SEISMIC EVALUATION

Commissioned by:

**Golden Gate Bridge, Highway
and Transportation District**
San Francisco, California

Prepared by:

T. Y. Lin International

In professional collaboration with:

Imbsen & Associates, Inc.

and

Geospectra, Inc.

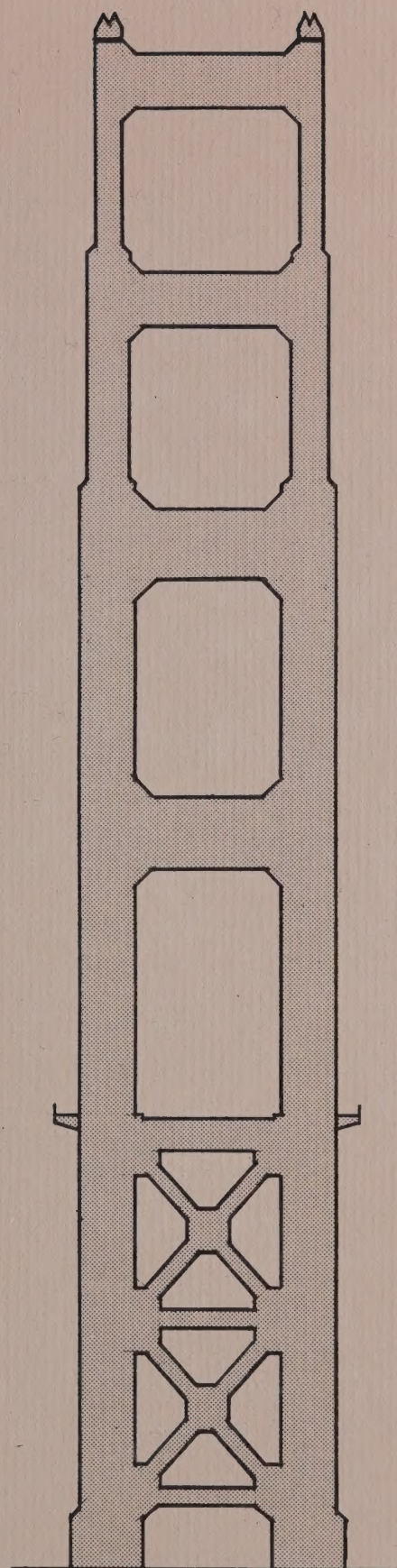
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by Mark A. Ketchum and Charles Seim

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San Francisco, California

Prepared by

T. Y. Lin International, San Francisco, California

in professional collaboration with

Imbsen & Associates, Inc., Sacramento, California

and

Geospectra, Inc., Richmond, California

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EXECUTIVE SUMMARY

The Golden Gate Bridge is a key component of the only highway artery connecting San Francisco with the counties to its north. The bridge was designed and built in the 1930's, prior to the advent of modern seismic engineering.

The bridge has performed well in previous earthquakes. The 1989 Loma Prieta earthquake caused severe damage to many structures, but caused essentially no damage to the Golden Gate Bridge. Shortly after that earthquake, the Golden Gate Bridge, Highway and Transportation District commissioned an evaluation of the seismic vulnerability of the famous span and its approach viaducts.

The seismic evaluation included the following components:

1. Review of the original design, analysis, and construction of the bridge with respect to seismic resistance.
2. Review of previous seismic studies and retrofit measures performed since the original design, analysis, and construction of the bridge.
3. Identification of seismic hazards at the bridge site.
4. Determination of probable ground motions at the site during different magnitude earthquakes.
5. Detailed analysis of the bridge under these ground motions.
6. Assessment of the performance and safety of structural components.
7. Identification of structural deficiencies.
8. Identification of possible seismic upgrade measures which could potentially correct the identified structural deficiencies.
9. Review of the scope, methods, and findings of the evaluation by recognized authorities in seismology, structural dynamics, and earthquake engineering.

The evaluation covered the main bridge and its contiguous approach viaducts, consisting of about 1.75 miles of structures. The structures evaluated include the six-span south approach viaduct and the five-span north approach viaduct, the south and north anchorage housings, the Fort Point arch, the pylons, and the three-span main suspension bridge.

SEISMIC HAZARDS AND GROUND MOTION

Primary seismic hazards to the bridge site include fault ruptures on nearby segments of the San Andreas fault system and the Hayward fault system. At their points of closest proximity, the San Andreas fault is about 7 miles west of the bridge in the Pacific Ocean, and the Hayward fault is about 10 miles east of the bridge in the Berkeley Hills.

Earthquakes of Richter magnitude 7 to 7.5, causing strong ground motion at the bridge site, could be generated by ruptures 60 to 120 miles in length on nearby segments of either fault system. Earthquakes of Richter magnitude 8 or greater, similar to the 1906 event, causing very strong ground motion of long duration at the bridge site, could be generated by a rupture about 250 miles in length on the nearby segments of the San Andreas fault.

Ground motions from a seismic event comparable to the 1906 San Francisco earthquake were synthesized by mathematical modeling of a rupture propagating past the bridge site on the San Andreas fault. The ground motion synthesis included the effects of site geology and the interference of seismic waves as a result of the rupture front moving along the fault.

BRIDGE ANALYSIS

Detailed mathematical computer models of the bridge were developed for evaluation of the behavior of the bridge and its various structural components. The computer models were formulated with particular attention to their ability to accurately capture dominant modes of behavior under earthquake ground motion, and to identify seismically vulnerable components.

Separate linear and nonlinear mathematical models of the main suspension bridge and the approach structures were used. The influences of the variation in ground motion over the length of the bridge, and of changes in structural behavior under long-duration ground motion were considered.

VULNERABILITY

Vulnerable components of the bridge were identified by detailed review and evaluation of the computer analyses. The degree of vulnerability varied greatly between the various structural subsystems of the bridge.

The bridge approaches, consisting of the Fort Point arch and the steel girder and truss viaducts supported on braced frames, are among the most vulnerable parts of the bridge. In a Richter magnitude 8 or greater earthquake centered near the bridge, high levels of damage can be expected in these structures. After such an earthquake, it is expected that these viaducts would no longer be able to carry traffic. There is substantial risk of collapse or impending collapse after such an event. The pylons and anchorage housings, of reinforced concrete construction, would experience damage similar to that of the approach viaducts. The probable high level of damage to these structures would also affect the performance of the main bridge. In a Richter magnitude 7 to 7.5 earthquake centered near the

bridge, damage would be somewhat less severe, but significant repairs would still be required.

The 6450-foot long suspension bridge is also vulnerable to damage during a Richter magnitude 8 or greater earthquake centered near the bridge. Critical components include the reinforced concrete piers supporting the steel towers, the bases of the towers above where they are anchored to the piers, the connections between the towers and the stiffening trusses, the saddles connecting the main cables to the tower tops, and the chords of the side-span stiffening trusses. The cable tie-downs in the pylons are also quite vulnerable, and their damage could lead to further secondary damage in the rest of the bridge. Most other structural components of the main bridge are in less danger, but the roadway expansion joints could be subject to secondary damage.

With the bridge in its current state, a Richter magnitude 8 or greater earthquake centered near the bridge, similar to the 1906 event, would likely cause some damage over the entire length of the bridge, with the risk of collapse of parts of the approaches. In the event of a collapse, this vital transportation link would be out of service for an extended period.

A Richter magnitude 7 to 7.5 earthquake on the Hayward or San Andreas fault, similar to the Loma Prieta event but centered near the bridge, would cause somewhat less damage, but could also result in temporary closure of the bridge during repairs.

RECOMMENDATIONS

While the Golden Gate Bridge has performed well in previous earthquakes, it is vulnerable to damage in a Richter magnitude 7 or greater earthquake with epicenter close to the bridge, and it could be closed for repair for some time after such an earthquake. The Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake has recommended that transportation structures of regional importance, such as the Golden Gate Bridge, should be retrofitted so that such closure is not required. This Board also recommended instrumenting such structures to record ground motion and structural response. Both these measures should be undertaken.

Structural retrofitting of the bridge prior to a strong earthquake could minimize the potential for damage and could eliminate the need for long-term closure for repairs after the earthquake. Retrofitting of all the vulnerable components would be required to minimize the risk such an earthquake poses. The retrofit measures can be built without closure of the bridge to traffic, using methods similar to those used during the recent deck replacement project.

The steel approach viaducts need to be made two to six times stronger with respect to seismic resistance to provide confidence that the bridge would remain open after a Richter magnitude 7 or greater earthquake centered near the bridge. The most critical components for retrofit are the foundations, supporting frames, structural bearings supporting the trusses, and the expansion joint restrainers. The trusses supporting the roadway also require strengthening, but are somewhat less critical.

The reinforced concrete anchorage housings and pylons need a similar level of strengthening. Such strengthening could be similar to the seismic reinforcement of older buildings in San Francisco, consisting of strengthening the foundations, building new internal framing, and bracing the exterior walls.

The main suspension bridge requires strengthening of its reinforced concrete piers and lower tower sections, restraint of the tower-pier connections, strengthening of the side span stiffening trusses and the connections between these trusses and the towers, strengthening of the connections of the cable saddles to the tops of the towers, and extending the tie-downs in the pylons to a more secure anchorage in the underlying rock. Additional structural modifications would alleviate secondary damage due to large movements of the bridge during the earthquake.

An order-of-magnitude cost of about \$75- to \$100- million (about ten percent of the replacement cost of the bridge) is being estimated for the complete strengthening of the Golden Gate Bridge and its approach viaducts.

CREDITS

This seismic evaluation was commissioned by the Board of Directors of the Golden Gate Bridge, Highway and Transportation District in November and December 1989. District Engineer Daniel E. Mohn guided and supervised the consultants' efforts, provided his knowledge of the bridge and its history, and participated in the review process. He and Deputy District Engineer Merv Giacomini also provided liaison between the District and the consultants.

T. Y. Lin International is the prime consultant to the District for the seismic evaluation. Principal-In-Charge Charles Seim assembled and directed the team, and provided liaison with the District. Project Manager Dr. Mark Ketchum performed or supervised technical analysis and engineering efforts. Project staff includes Ron Zimmerman, Al Heldermon, Kar Seah, Dr. Albert Tung, and J. T. Teng. Technical advice was provided by Professor Alexander Scordelis and Professor Hassan Astaneh, both of the University of California at Berkeley. Special thanks are due to Professor Emeritus Frank Baron, of the University of

California at Berkeley, who provided insightful remarks as well as a rich history of previous work on the Golden Gate Bridge.

Imbsen & Associates, Inc. was subconsultant assisting in the seismic evaluation of the main suspension bridge. Dr. Roy Imbsen and Dr. W. David Liu are responsible for IAI's contributions. Project staff includes F. S. Nobari and Dr. R. Hamidi.

Geospectra, Inc. was subconsultant for seismic risk and ground motion evaluation. Dr. J. P. Singh and Dr. Mansour Tabatabaie are responsible for Geospectra's contributions.

A board of review, consisting of recognized experts in the fields of seismology, structural dynamics, and earthquake engineering, provided an independent evaluation of the scope, methods, and findings of the evaluation. This board of review consists of Professor Bruce Bolt, Professor Joseph Penzien, and Professor Vitelmo Bertero, all of the University of California at Berkeley.

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1. INTRODUCTION

The Golden Gate Bridge is a key component of the only highway transportation artery connecting San Francisco with the counties to its north. The bridge was opened to traffic in 1937 after ten years of planning and four years of construction, and was the longest span structure in the world until 1964. Today the bridge is considered to be one of the most significant construction achievements of the early 20th century and is recognized around the world as a symbol of San Francisco.

The bridge was designed and built using the most advanced structural technology of the time. The construction technology was also based on the standard practices of the time, and in some areas set standards for many years following. Some details of the construction, however, are no longer used in regions of high seismicity. The structural analysis theories that were used in the design of the bridge allowed a reasonably accurate evaluation of stresses and deflections under self weight, traffic loads, and other static loads. They did not, however, accurately predict the response of the bridge to dynamic loads such as wind and earthquake. The simplified approaches to the analysis of the bridge have since been demonstrated to be inadequate for evaluating the true response of the bridge to these dynamic effects. The response of the bridge to wind has been critical on several occasions, so that retrofit measures have been incorporated to improve its wind resistance. No significant seismic damage to the bridge has ever been detected, and no comprehensive evaluation of its seismic resistance had been commissioned before 1989.

For fifteen seconds, at 5:04 p.m., October 17, 1989, the San Francisco Bay area was shaken by the "Loma Prieta" earthquake, measured at 7.1 magnitude on the Richter scale and centered about 80 miles south of the bridge site in the Santa Cruz mountains. For months thereafter, the San Francisco Bay area continued to feel strong economic distress as vital vehicular transportation links remained closed from damage suffered during this brief and fairly distant earthquake. While the Golden Gate Bridge suffered no observed damage in this earthquake, it was recognized that stronger earthquakes, centered closer to the bridge, could be destructive to the structure.

Immediately following the Loma Prieta earthquake, the Golden Gate Bridge, Highway and Transportation District engaged T. Y. Lin International to evaluate the vulnerability of the Golden Gate Bridge to several likely fault rupture scenarios including the expected maximum credible earthquake. The seismic evaluation work was performed under an addendum to T. Y. Lin International's consulting services agreement for the Golden Gate Bridge Mass Transit

Feasibility Study. Subconsultants Imbsen & Associates and Geospectra were engaged to provide their specialized expertise in structural analysis and determination of ground motion to the evaluation.

This report discusses the engineering issues and background material studied in the seismic evaluation; identifies ground motion criteria for likely fault ruptures in the San Francisco Bay area; identifies structural performance criteria; presents the structural modeling used for the seismic analysis; describes the vulnerability to seismic excitation of the suspension bridge and the approach structures of the Golden Gate Bridge; presents recommendations for appropriate action including seismic upgrade, as well as a very approximate estimate for the cost of the upgrade construction.

At about the same time that the Board of Directors of the Golden Gate Bridge, Highway and Transportation District directed this evaluation to be performed, Governor Deukmejian appointed a Board of Inquiry to investigate the causes and effects of the damage sustained in the Loma Prieta earthquake and to make recommendations for strengthening existing structures for the expected event of a major seismic disturbance in the San Francisco Bay area in the next decade. After this Golden Gate Bridge evaluation was well under way, the Board of Inquiry, chaired by Professor George W. Housner, issued the report *Competing Against Time, Report to Governor George Deukmejian from The Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake* on May 31, 1990 [1], which summarized the findings of the Board. Some of the findings of the Board of Inquiry have a direct bearing on this study and are incorporated into this report.

1.1 BRIDGE DESCRIPTION

The 9151 ft overall length of the Golden Gate Bridge consists of a number of different structure types. A plan and elevation of the bridge are shown in Fig. 1.1. The approach viaducts are of steel girder, truss, and arch construction; the anchorages, anchorage housings, pylons, and piers of the main structure are of reinforced concrete construction; and the superstructure and piers of the landmark main suspension bridge are of steel construction. The bridge has been modified several times over its 53 year history; its current configuration is described below.

The main structure across the strait consists of three suspension spans - a center span of 4200 ft and two side spans of 1125 ft each - suspended from two 36.5 in. diameter steel wire cables spaced 90 ft apart. The cables are each supported on two steel towers and are anchored in mass concrete anchorage blocks at their

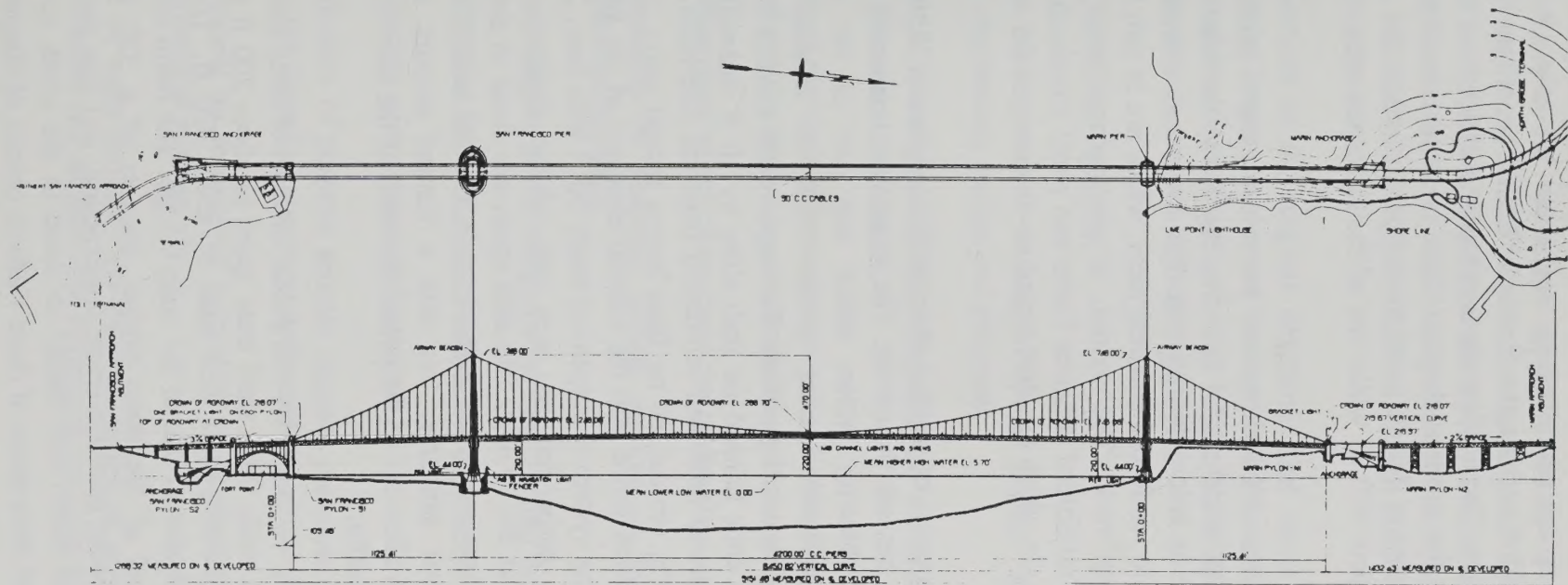


Fig. 1.1: Bridge Plan and Elevation (Strauss, 1937 [1])

ends. At the shoreward end of each side span the cables pass through reinforced concrete pylons where they are restrained by steel cable "tie-downs." The tie-downs are designed to hold the cables at a fixed elevation from where they start to rise toward the tower tops. The north anchorage block is 215 ft from the north concrete pylon tie-down; the south anchorage block is more remote from the south concrete pylon tie-down at 583 ft. In its current configuration, the drape of the main span cables is about 470 ft, while that of the side span cables is about 75 ft.

The steel towers, each rising 746 ft above low water level, are of multicellular plate construction, with horizontal portal strut bracing above the roadway and double diagonal cross bracing below the roadway. The towers are anchored with dowels and riveted angles to reinforced concrete piers, which are in turn founded directly on the underlying rock. The pylons, at the extreme ends of the suspension bridge, are of reinforced concrete frame and panel construction with an open box configuration, in which the steel strand tie-down ropes are anchored just below deck level.

The suspended structure consists of parallel-chord stiffening trusses, 25 ft deep, spaced 90 ft apart directly beneath the cables. The trusses are connected to each other with a top lateral bracing system which was part of the original construction, and a bottom lateral bracing system which was installed in the 1950's, forming a trussed-tube configuration. The upper deck consists of a six-lane, 62 ft wide highway deck flanked on both sides by 10 ft sidewalks. The orthotropic steel plate highway deck and reinforced concrete sidewalks, which were installed in the 1980's, are carried on floor beams spaced every 25 ft (at every truss panel point) which frame into the vertical elements of the stiffening trusses. The entire stiffening system is suspended every 50 ft with four, 2 11/16 in. diameter bridge rope suspenders on each side of the roadway. These suspenders were replaced in the 1970's. The side spans are fixed to the tower with joints that allow only rotations about vertical and horizontal axes. The main span is fixed to the tower similarly, but with a limited amount of free longitudinal movement also allowed. At the pylons the ends of the side spans are also free to move longitudinally.

The south approach viaduct, starting at the toll plaza, consists of three steel plate girder spans of about 71 ft each, three steel truss spans of about 200 ft each, a reinforced concrete pylon, and a steel arch span of about 320 ft. The deck framing of this viaduct is similar to that of the main bridge. The north approach viaduct, starting at the end of the main bridge, consists of a 320 ft long anchorage housing of reinforced concrete frame construction, and five steel truss spans of about 200 ft each which are similar to those in the south approach viaduct. The steel girder and truss spans of these viaducts consist of simple spans

supported on braced frame bents. The superstructures are externally statically determinate, with no redundant supports, which is typical of steel bridge construction of the 1930's. The arch span is stiffened by a trussed arch-rib; what appears to be deck stiffening is an architectural treatment only. In about 1980, the approach viaducts were seismically retrofit with joint restrainers which are similar in concept to those of the Caltrans retrofit program of that time, and with foundation improvements.

All connections in the original structural steel were made with rivets. The bottom lateral bracing system of the suspended structure includes both riveted and bolted connections. During normal maintenance, some of the original rivets were replaced with modern high-strength bolts.

1.2 SCOPE OF WORK

The scope of this seismic evaluation of the Golden Gate Bridge included the following engineering studies and analyses:

- a. Review of the original design, analysis, and construction of the bridge with respect to seismic resistance.
- b. Review of previous seismic studies and retrofit measures performed since the original design, analysis, and construction of the bridge.
- c. Identification of seismic hazards at the bridge site.
- d. Determination of probable ground motions at the site during different magnitude earthquakes.
- e. Detailed analysis of the bridge under these ground motions.
- f. Assessment of the performance and safety of structural components.
- g. Identification of structural deficiencies.
- h. Identification of possible seismic upgrade measures which could potentially correct the identified structural deficiencies.
- i. Peer review by recognized authorities in seismology, structural dynamics, and earthquake engineering.

The scope of work excludes any detailed design of seismic upgrade measures, and also excludes any detailed cost estimating for these measures.

1.3 ORGANIZATION OF REPORT

This report presents all aspects of the scope of work with related discussion and interpretation of data. The presentation is divided into the following components:

- a. Background review of the history of the bridge structures including their original design, analysis, and construction, as well as structural modifications made in the intervening years; a review of some of the previous investigations of the bridge including vibration studies and seismic evaluations; and a review of pertinent aspects of the report *Competing Against Time, Report to Governor George Deukmejian from The Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake* on May 31, 1990 [1].
- b. Ground motion studies, consisting of a review of geological data, seismic risk analyses, determination of response spectra for various intensities of earthquakes, and determination of ground motions for deterministic analysis of the structures.
- c. Structural performance criteria developed for the evaluation of bridge performance, including structural subsystems and components, in order to determine the level of damage expected and the extent of retrofit required for these subsystems and components for the various intensities of earthquakes.
- d. Structural computer models and structural response evaluations of the models to evaluate the response of the structures to the ground motions.
- e. Determination of the seismically vulnerable components found in each of the structural subsystems making up the bridge.
- f. Preliminary recommendations for remedial actions which should be undertaken to upgrade the seismic resistance of the bridge and to increase its ability to withstand earthquake ground motions without significant damage.
- g. Recommendations for additional engineering work and reviews to be undertaken so that necessary and sufficient seismic upgrades can be incorporated in the bridge as quickly as possible.

2. BACKGROUND TO THE PRESENT STUDY

A review of some of the documents relating to the design, construction, and fifty-three years of operation of the Golden Gate Bridge, as well as various investigations that have been performed on the Golden Gate Bridge, provides a background and a basis for the analyses and conclusions of this seismic study.

2.1 ORIGINAL DESIGN AND CONSTRUCTION

In 1921, Joseph Strauss proposed a hybrid cantilever suspension bridge across the Golden Gate and used this concept for his promotion of the bridge until 1924-25, through the granting of the initial War Department permits, and the initial efforts to bring into existence the Golden Gate Bridge District. The design underwent numerous modifications, with a suspension design replacing the original Strauss design in 1925. There followed continuous revisions and modifications in the design even after creation of the District had been approved and funding had been secured. It was not until 1930 that the final length of the span was determined at 4200 ft and a cable spacing of 90 ft was selected. The original planning, design, final analysis and construction of the Golden Gate Bridge is chronicled in Joseph Strauss' *Report to the Chief Engineer to the Board of Directors*, [2] which was largely written by his assistant, Clifford E. Paine. The structural systems of the bridge, as they were built and modified to date, are briefly described in Chapter 1 of this report.

The bridge was designed using the most advanced theories of suspension bridge behavior that had been formulated at the time. These theories allow a reasonably accurate calculation of stresses and deflections under self weight, traffic loads, and other static loads. Earthquake and wind forces were considered as horizontal static effects although their actions are actually dynamic. The Golden Gate Bridge engineers designed the bridge conservatively, making use of high estimates of traffic loads and low values of allowable stresses in the primary structural components. The result is a bridge which is quite flexible and which has a high reserve in vertical load capacity.

The original analysis and design of the bridge for seismic effects is described in the published paper "Provision for Seismic Forces in Design of Golden Gate Bridge," by Consulting Engineer Leon S. Moisseiff [3]. This paper, originally presented at the 1939 Annual Convention of the American Society of Civil Engineers, presents the philosophy and calculations used by the original engineers. The following quotations from this paper illustrate the approach of the design engineers and their resolution of key issues:

On general assumptions of seismic activity and loads:

"Planning of structures to sustain the effects of earthquake without undue distortion and rupture is still in a tentative and crude stage. This is not because engineering knowledge is insufficient to cope with the proper design of structures for known forces, but because the data on earthquakes are as yet meager and the knowledge of their effects is still vague." [3, p. 33]

"Records of destructive earthquakes show relatively small amplitudes -- seldom exceeding 1.5 in. -- and a minimum period of vibration of approximately 1.35 sec." [3, p. 33]

"To evaluate the effect of earthquakes in the San Francisco region for the Golden Gate Bridge, an amplitude of 1.24 in. and a period of 1.3 sec. were assumed. for these values a maximum acceleration of 2.42 ft per sec per sec -- that is, 0.075g -- results." [3, p. 33]

"It is assumed that all parts of the structure are sufficiently connected to each other to assure full interaction during the earthquake." [3, p. 34]

"The natural frequency of the superstructure is between 5 and 25 sec, depending on the direction of earthquake movement. The frequency of a destructive earthquake can thus never be in resonance with that of this structure or any part of it. It was therefore considered justifiable to translate the effect of earthquake into static forces acting on the structure." [3, p. 33]

"The greatest acceleration assumed for the superstructure and the anchorages is 0.075g; for the main piers this was, however, increased to 0.10g. A solid body which does not elastically deform would thus be subjected to a horizontal force of 7 1/2 to 10% of its weight acting at the center of gravity. This is true for the piers and anchorages but will not be fully applicable to the superstructure because of the flexibility of the latter. The acceleration exerted on the ground will lose itself gradually in deflections of the structure." [3, pp. 33-34]

On earthquake waves acting parallel to the longitudinal axis of the bridge:

"Suspended Span. The suspended center span is provided at each end at the towers with expansion joints. It can thus move longitudinally and its weight cannot be accelerated by an earthquake. No forces are

therefore produced on the towers by an earthquake acting in a longitudinal direction." [3, p. 35]

"*Towers.* Studies indicate that the flexibility of the steel towers reduces the initial acceleration to about half. The effect of the earthquake is thus equivalent to a static force of 3.75% of the weight of the tower acting horizontally." [3, p. 35]

On earthquake waves acting transversely to the longitudinal axis of the bridge:

"*Suspended Span.* Because of the great length of the suspended spans, only part of this mass is accelerated." [3, p. 35]

"*Towers.* ... the lateral stiffness was assumed to be so high that the acceleration due to earthquake is extended over the full length of this structure. ... For further safety, this force was assumed to act at mid-height of the tower." [3, p. 35]

On overall conclusions with respect to seismic design and resistance:

"In conclusion it may be pointed out that the effect of earthquake on the deflections and stresses in the superstructure was found to be considerably less than that of a 30-lb wind acting on the bridge. The towers are firmly anchored to the piers, and no danger of movement of the tower bases exists." [3, p. 35]

These observations by one of the original consulting engineers for the design of the bridge indicate that the basis of the seismic design of the Golden Gate Bridge was not in keeping with modern practice.

2.2 STRUCTURAL MODIFICATIONS

Over its 53-year history, the Golden Gate Bridge has been structurally modified on several occasions. These modifications were performed to provide an increase in wind resistance, to correct some structural deterioration problems observed in some of the elements of the bridge, and to provide improved seismic resistance in the approach viaducts.

2.2.1 Wind Retrofit

At the time the Golden Gate Bridge was designed and built, wind was treated analytically as a static lateral load. Tools for analysis of this static wind load and other static loads were sufficiently accurate; wind load stresses were

demonstrated to be fairly small. However, the flexibility of the design under the dynamic effects of wind results in a structure that experiences deck oscillations in strong wind storms.

The modern science of suspension bridge aerodynamics was born after the Tacoma Narrows Bridge collapsed in a moderate wind storm in 1940, only three years after the Golden Gate Bridge was completed. Unstable oscillation, similar to that of the Tacoma Narrows Bridge but not as severe, occurred on the Golden Gate Bridge in a strong wind storm in December 1951. The oscillations of the deck in that storm caused the bridge to be closed to traffic for a few hours in the interest of traffic safety. Some structural damage also occurred.

After the 1951 storm, a wind engineering investigation was carried out by Clifford E. Paine, O. H. Ammann, and Charles E. Andrew [4]. On the basis of their recommendations, the bridge was retrofitted in 1954 with a bottom lateral bracing system in the suspended structure. The bridge has performed more satisfactorily in wind storms since that time. This retrofit is described in Paine's Supplement [5] to the Strauss chronicle of the bridge's construction.

A wind stability evaluation was performed in 1961 as a part of a study of the engineering feasibility of adding rapid transit facilities to the bridge. This study [6], performed by Steinman, Boynton, Gronquist & London Consulting Engineers for the Bay Area Rapid Transit District, concluded that the ability of the bridge to resist wind did not meet standards then current, and that the ability of the bridge to resist wind could be improved using one or more of several alternative measures.

The bridge was again closed to traffic in wind storms in 1981 and 1982, however no damage occurred on those occasions.

Another wind engineering evaluation was performed in 1990 as a part of a transit feasibility study. This study [7], performed by T. Y. Lin International for the Golden Gate Bridge, Highway and Transportation District, concluded that the bridge in its current configuration does not possess as great a margin of safety against wind instability as modern practice would normally require. Thus, the Golden Gate Bridge remains a wind-critical structure.

2.2.2 Suspender Replacement

In 1968 some of the original suspenders on the bridge were found to be corroded and a decision was made to replace all the original suspenders. This replacement was effected between 1973 and 1976. The replacement suspenders are nominally stronger than the original ones.

2.2.3 Seismic Retrofit

After the 1971 San Fernando Valley earthquake, Caltrans initiated a seismic retrofit program for state-owned bridges. The first phase of this program consisted of restraining the expansion joints with cables, so that in the event of large structural displacements, the joints would be restrained from physical separation.

The Golden Gate Bridge, Highway and Transportation District retrofitted the approaches of the Golden Gate Bridge using the Caltrans guidelines in 1982. The bearings and expansion joints of the girder and truss approach viaducts were fitted with threaded bar restrainers, evidently intended to prevent excessive relative displacements and to provide backup strength capacity. The Fort Point arch span was fitted with vertical restrainers at the bearings supporting the springline of the arch ribs and with transverse restrainers beneath the roadway, evidently intended to prevent rigid-body toppling of the arch. The approach-viaduct foundations were also strengthened. In keeping with the Caltrans policy at that time, no overall upgrade of the seismic resisting system was performed.

2.2.4 Deck Replacement

In 1980 the original concrete roadway slab on the suspension spans and on the steel approaches had deteriorated to such an extent that replacement was required. A new decking system was designed, and between 1981 and 1985 the original concrete deck slabs were removed and replaced with a lightweight orthotropic steel deck system two ft wider than the original. The original concrete sidewalks were also replaced. This replacement resulted in a net reduction in weight of the suspension bridge of about eleven thousand tons.

2.3 VIBRATION MEASUREMENTS

Measurements of the vibrations of the Golden Gate Bridge have been made on several occasions over the life of the structure.

2.3.1 Early Studies

Between 1933 and 1942, at several stages of construction and after completion, seismological instruments were set up on the Golden Gate Bridge by the U.S. Coast and Geodetic Survey on the piers, towers, cables and deck to measure any vibration that might occur. In 1942 a Hall Recorder accelerometer was installed at the middle of the main span on the sidewalk; this accelerometer was replaced in 1945 by new instruments designed to record only vertical vibrations.

Following the failure of the Tacoma Narrows Bridge in 1940 the Golden Gate Bridge, Highway and Transportation District entered into a cooperative venture with the Bureau of Public Roads to install ten instruments for measuring vertical movement of the bridge. These instruments were operated by the Bureau of Public Roads until 1950 when the venture was terminated. The District continued to operate the instruments until 1954.

2.3.2 Abdel-Ghaffar and Scanlan Measurements, 1982

In 1982, a team of researchers assembled by the Civil Engineering Department of Princeton University and Kinematics Inc. of Pasadena, California, made a comprehensive series of measurements of the ambient vibration properties of the Golden Gate Bridge. One purpose of these measurements was to determine the three-dimensional mode shapes and the associated natural frequencies and modal damping factors. The methods and findings of the investigations are summarized in two technical papers by A. Abdel-Ghaffar and R. H. Scanlan [8][9].

Reduction of the data provided by these measurements allowed the identification of frequencies, mode shapes, and modal damping factors for 73 vibration modes in the suspended structure and 12 vibration modes in each tower. These vibration properties were used as the basis for calibrating the analytical computer models for the seismic response analyses of the present investigation.

2.4 PREVIOUS SEISMIC EVALUATIONS

A thorough seismic evaluation of the Golden Gate Bridge had not been commissioned between the date of its original design and the time this study was initiated. The bridge has been used frequently as an example problem in dissertations and technical papers on the topic of seismic response evaluation of suspension bridges, but these evaluations generally have not provided any evaluation of response beyond the presentation of numerical results.

Although the approach viaducts were upgraded with joint restrainers, the restrainer design was typical of its era, with emphasis on "the performance of individual elements (restraining motion at expansion joints)" and "did not consider the response of the whole structure" [1, p. 36]; therefore this design data were not reviewed here as a previous seismic evaluation.

2.4.1 1976 Study by Professor Baron

In November 1976 a research report entitled *The Effects of Seismic Disturbances on the Golden Gate Bridge* [10], by Professor Frank Baron and his Research Assistants Metin Arikan and Raymond E. Hamati, was published by the

Earthquake Engineering Research Center at the University of California at Berkeley. The studies performed in preparation of the report were intended "to supplement the studies made by others during the design phase of the bridge" [10, p. 1.1], and incorporated then-state-of-the-art determinations of ground motion and structural response. The writers found that "the findings reported ... are important and merit attention" and that "they are presented for their possible value to the profession of structural engineering and to those responsible for the operation and maintenance of the bridge." [10, p. 1.5].

While Professor Baron's report was not commissioned by the Golden Gate Bridge District, and while its basic intent was evidently not to explore in ultimate detail the possible structural consequences to the bridge of a major earthquake, the report did conclude that certain elements of the bridge could sustain serious damage during an earthquake of Richter magnitude 8.+.

Professor Baron's studies were directed to the main suspension bridge only; no evaluation of the approach viaducts was undertaken. The observations and conclusions of the Baron report of most interest to the present investigation include the following primary points:

On the ground motions used for the analysis:

Five cases of propagating ground motions are considered for determining the effects of multiple support excitations on the dynamic response of the Golden Gate Bridge. In each case, the bridge is subjected to a ground motion which travels along the bridge with a constant velocity exciting the supports in the longitudinal direction. It is assumed that the given ground motion does not change its characteristics while it propagates along the bridge; that is, the supports of the bridge experience the same ground motion but at different times. ... Mention however need be made that serious inadequacies arise in propagating records of ground motion that have been obtained at isolated points." [10, p. 6.12]

On the effects of multiple-support excitation:

"The values of maximum response associated with multiple-support excitation are significantly different from those obtained through spectral or time-history analyses for uniform-support excitation. For multiple-support excitation, the maximum values of many parameters are considerably bigger than those obtained for uniform-support excitation. The latter is particularly true for (a) the magnitudes of the cable forces at the anchorages, (b) the moments at the bases of the

towers, and (c) the moments at an intermediate elevation of the tower. In propagating the SI8+ record at different speeds, it is observed that the values of maximum response are in general increased as the velocity of the record is made smaller. It is also observed that the value of the parameters are practically insignificant when the analysis is conducted only for the quasi-static propagation of the SI8+ ground motion. On the basis of this observation, it is concluded that the big values are mainly due to the dynamic effects rather than the quasi-static effects of multiple-support excitations." [10, p. 7.3]

On the inadequacy of the standard seismic coefficient for analysis the bridge:

"A static analysis of the bridge, based on the use of a seismic coefficient of 0.05g for the determination of lateral force yields an inadequate description of the maximum values of dynamic response that can be produced by a strong earthquake of Richter magnitude 8.+." [10, p. 7.6]

On the inadequacy of wind stability as a criterion for seismic safety:

"Suitable provisions for aerodynamic stability and for the dynamic effects of wind do not necessarily insure the safety of a suspension bridge to withstand seismic disturbances. In addition to inputs, the responses and the possible modes of failure are different for the two kinds of excitation." [10, p. 7.7]

On the effects of a moderate earthquake:

"The effects of a moderate earthquake on the Golden Gate Bridge are essentially negligible." [10, p. 7.4]

On the effects of a strong earthquake:

"A strong earthquake of Richter magnitude 8.+ can produce big values of stresses in certain elements of the bridge." [10, p. 7.4]

"In general, the elements of the stiffening system and the suspenders are not affected much by the various kinds of ground excitations that are considered in these studies." [10, p. 7.4]

"However, the elements of the main-cables, particularly those in the side spans, are affected appreciably by multiple-support excitations. ... Further, attention is called to the details that are employed in

anchoring the various strands of the cables at the anchorages. Obviously, the safety of the bridge as a whole is dependent as much on the structural integrity of the anchorage details as it is on the strength of the main-cables." [10, pp. 7.4, 7.5]

"For multiple-support excitations, the maximum values of the calculated stresses indicate that the deformations at the bases of the towers can be well into the yield range for severe earthquake excitations. ... Based on these values, the outer cells of the towers would buckle when in compression, and yield or fracture at splice plates when in tension. ... At an intermediate section which lies approximately at the same elevation as Strut 3, we calculate maximum values of stresses in the extreme fibers ... [indicating] the possibility of damage by buckling of the plates in the outer cells." [10, pp. 7.5, 7.6]

Concluding Remarks:

"The study of the Golden Gate Bridge indicated that studies of other suspension bridges located in areas of large seismicity need take into account the influences of earthquake excitations in the transverse directions as well as in the longitudinal directions of such bridges. Further, special considerations need be given to the effects of multiple-support excitations. The latter kinds of excitations tend to increase the participation of the higher modes in the total response, leading to bigger values of maximum response than those obtained for uniform-support excitations. In any case, the proper modeling of a suspension bridge is required to obtain an adequate description of the various kinds of vibrations that can occur." [10, p. 7.7]

The report of Professor Baron was presented to the District and to the public at the time of its publication, and was subsequently reviewed for the District by Amman & Whitney, Consulting Engineers.

2.4.2 Ammann & Whitney Review of the Baron Report

In May 1977, the Golden Gate Bridge, Highway and Transportation District commissioned a review of Professor Baron's report performed by Ammann & Whitney Consulting Engineers in New York. The review was completed in December 1977 [11].

The major points of the review include the following excerpts:

"In calculating stresses in this section [Section E-2 of the tower], Professor Baron has correctly used the cross-sectional area of this section, but has applied to it the moment arm of the base section (26.29 ft) instead of the distance applicable to section E-2 (22.79 ft). The appropriate correction of his calculation reduces the maximum stresses from 54.2 to 51.6 ksi compression, and from 39.6 to 36.6 ksi tension." [11, p. 4]

"Professor Baron has [incorrectly] assigned a value of "infinite" to the Moment of Inertia of the 32 ft high base section [of the tower]. [11, p. 4]

"Professor Baron's analysis is based on the assumption of absolute fixity at bottom of the tower, resulting in a moment of 3,916,650 ft. kip. at the base of tower. This is also the moment value used by Professor Baron for the stress calculation in Section E-2, 32 ft above the base. ... [More realistic assumptions would result] in further stress reduction to below yield strength level." [11, p. 4]

"The assumption of complete fixity at bottom of the tower is basic to Professor Baron's stress analysis. Actually, complete fixity cannot be attained. The tower base section is connected to the concrete pier by means of structural angles embedded in the concrete. The moment capacity of this angle connection is considerably less than the capacity needed to transfer the moment used in Professor Baron's calculations from the tower base to the concrete pier. Therefore, the assumption of a moment of this magnitude is unrealistic and the entire stress analysis on that basis becomes questionable." [11, p. 4]

"The analyses performed were on an elastic basis with no apparent consideration for plastic action and acceptable ductility ratios. ... some allowance for plastic action seems reasonable." [11, p. 8]

"A spectra reduction factor equal to 4 is used [in the Caltrans 1973 Earthquake Design Criteria for Bridges] to account for plastic action. Another reduction factor considered in the criteria is a "risk factor" which varies from a value of 2 to 1, depending on the period and importance of the structure. ... [These reductions] could easily result in reducing the tower stresses calculated by Professor Baron to acceptable levels." [11, p. 9]

The report concluded with the following observations:

"Professor Baron's study indicates stress conditions in parts of the main tower as exceeding yield or buckling limits, causing him to be fearful of possible local failures. Our review has found certain misinterpretations in his buckling strength assumptions and in his stress calculations, as a result of which stresses calculated in accordance with his theory would fall within acceptable limits.

"Furthermore, we have determined that the degree of end-restraint of the tower assumed by Professor Baron cannot possibly be attained, thus putting in question his entire moment and stress determination. In addition, it would seem reasonable to assume a more favorable spectra response due to plastic action of the tower than was used in Professor Baron's study.

"Consequently, it is concluded that tower stresses resulting from the ground motions used in this study would actually be considerably below those reported by Professor Baron and well within acceptable levels. It is, therefore, recommended that no structural modifications of the towers be considered and no further action be taken by the District in response to Professor Baron's report." [11, p. 10]

2.4.3 Analysis of Baron vs. Ammann & Whitney

The points at issue between Professor Baron's report and the Ammann & Whitney response were considered to be critical in this investigation and were re-evaluated in light of new technologies and analytical techniques.

a. Evaluation Technology

Professor Baron's seismic evaluation of the Golden Gate Bridge was based on 1976 state-of-the-art technology for determination of ground motion and for structural response analysis. The developments over the last fourteen years in the field of earthquake engineering permit evaluation of the strengths and deficiencies of the approaches Professor Baron used in his study.

Ground Motions

In evaluating the response of the bridge structures, both uniform support excitation and multiple-support excitations were considered. For the uniform support excitations, Professor Baron used two different sets of ground motions. The first set represented an earthquake of Richter magnitude 7.7 and consisted of recordings made at Taft Station during the 1952 Kern County Earthquake. The

Taft Station is a firm site and is located approximately 40 km from the epicenter. The second set corresponds to a strong earthquake of Richter magnitude 8+ and consisted of an artificial record generated by Seed and Idriss in 1969 known as the SI8+ ground motion. This record is considered representative of bedrock conditions and was developed by splicing portions of scaled and unscaled records from previous earthquakes.

Two important observations about the SI8+ record should be noted:

1. The SI8+ record showed large residual displacements at its end, which Professor Baron eliminated by adjusting the tail of the record.
2. Spliced records such as the SI8+ do not represent real ground motion. The displacement record of the Taft recording crosses zero ten times in thirty seconds, while the displacement record of the spliced SI8+ motion crosses zero only twice in sixty seconds.

For the multiple support excitation analysis, five cases of propagating ground motions were considered by Professor Baron. In each case, the bridge was subjected to ground motion which excited the supports in the longitudinal direction with the same ground motion but at different times. In three of the cases, the SI8+ record was propagated, and in the other two cases, sinusoidal waves were propagated. The deficiencies of this approach were recognized by Professor Baron.

The simplifications in ground motion modeling necessary at the time of Professor Baron's evaluation are unnecessary today. Since 1979, recordings of earthquakes by strong motion instrument arrays installed in California and Taiwan have provided a wealth of information on the variability of ground motion due to wave propagation. This information, when combined with computational methods now available, provides the means of generating ground motions that can properly modeled wave fields for seismic sources.

Structural Modeling and Response Analyses

Professor Baron use a two-dimensional model to study longitudinal and vertical response and a three-dimensional model to study transverse and torsional response. The excellent correspondence of the vibration properties of the bridge computed from Professor Baron's models [10, pp. 5.27-5.36] to those determined from subsequent measurements by Abdel-Ghaffar and Scanlan [8][9] serve to validate his models.

The response analyses used by Professor Baron were based on a theoretically complete evaluation of the multiple-support excitation problem of the suspension

bridge. Total response was considered as consisting of dynamic and quasi-static components. This approach is capable of providing a precise linear elastic analysis of the bridge to the support motions used.

The approaches used in Professor Baron's evaluation were theoretically complete and provided precise results for the linear elastic response of the structure, within the limitations of the ground motions he used. The theoretical approach used for seismic response analyses reported below differs from Professor Baron's only in the use of more detailed numerical models made possible by the increased capabilities of modern computers; in the use of more modern numerical implementations of the basic structural theory; in the local nonlinear modeling of the tower bases to more precisely evaluate the damage that might occur there; and in the improved ground motion assumptions.

b. Response and Damage Evaluations

The most significant damage predicted by Professor Baron in his report is in the plates at the base of the tower and in the tower legs between their bases and the elevation of the roadway. Amman & Whitney discredited this damage prediction on the basis of the invalidity of the assumption of base fixity of the towers and some arithmetic errors in stress calculations. However, the ramifications of the nonlinear behavior of the tower bases were not examined to their logical conclusion, and Ammann & Whitney in turn made some arithmetic errors in their stress calculations.

Additional damage is predicted in the links between the side spans and the towers by extrapolating from the moment diagrams that Professor Baron provides. Failure of the links, while relieving stresses in the towers, would cause major damage to the deck expansion joints and the vertical support links.

The extrapolated tower shears also indicate that the connections between the cable saddles and the tops of the tower may also be vulnerable. Extrapolation from the tabulated suspender forces indicates that a large percentage of the side span stiffening truss weight must be transferred from the main cables to the truss itself, with possible damage to those trusses as a result.

c. Conclusions

The analysis of the points raised by Professor Baron and his colleagues shows it is possible to conclude that several components of the Golden Gate Bridge would be endangered by a major earthquake. These components include the tower bases (particularly the tie-down assemblies), the towers themselves, the shear links

between the side spans and the towers, the tower saddles, and the side span stiffening trusses.

2.5 REPORT OF GOVERNOR'S 1989 BOARD OF INQUIRY

Following the October 1989 Loma Prieta earthquake, California Governor George Deukmejian appointed a Board of Inquiry to investigate the causes and effects of the damage sustained and to make recommendations for strengthening existing structures for the expected event of a major seismic disturbance in the Bay area in the next decade. This Board of Inquiry was chaired by George W. Housner and consisted of members Joseph Penzien, Mihran S. Agbabian, Christopher Arnold, Lemoine V. Dickinson Jr., Eric Elsesser, I. M. Idriss, Paul C. Jennings, Walter Podolny Jr., Alexander C. Scordelis, and Robert E. Wallace. In May 1990 the Board issued the report *Competing Against Time, Report to Governor George Deukmejian from The Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake* [1], some aspects of which are particularly pertinent to this evaluation.

The scope of the investigation by the Governor's 1989 Board of Inquiry into the Loma Prieta Earthquake covered all major transportation structures owned and operated both by the State of California and by special transportation districts. In the course of its work, the Board reviewed previous and present policies of Caltrans, along with those of the various special transportation districts, and made recommendations for both general, state-wide policy changes and for implementation of specific programs for structures such as the Golden Gate Bridge, the Bay Bridge, and the Bart Trans-bay tunnel.

Noting that "Other major earthquakes of equal or greater magnitude are also anticipated on the Hayward fault and other segments of the San Andreas fault -- much closer to population concentrations than was the Loma Prieta earthquake", [1, p. 21], the Governor's Board of Inquiry characterized the Loma Prieta experience as "a 'live' exercise demonstrating the [effects of the] short-term closure of a single cross-bay link,[1, p. 38]." The Board of Inquiry report further noted that: "For automobile traffic, the Golden Gate Bridge and Bay bridges are essentially nonredundant systems, with alternative routes via the other bridges being time consuming to a level that seriously impacts commercial and institutional productivity."

The Board noted also that "The design of the Bay and Golden Gate Bridges in the early 1930s employed the state-of-the-art in earthquake engineering at the time of their design. These structures are too important to wait until a future earthquake to discover if they perform unacceptably. Earthquake engineering

assessments should be completed to determine if their performance is acceptable and, if necessary, to determine how their seismic performance can be improved to an acceptable level." [1, p. 85]

The primary concern of the Board of Inquiry was the economic impact of the closure of major transportation facilities, such as the San Francisco-Oakland Bay Bridge and the adoption and implementation of state-wide policies with the specific goal to assure "that all transportation structures be seismically safe and that important transportation structures maintain their function after earthquakes." [1, p. 9] (Emphasis was added because of the many times the report returned to this basic performance criterion).

The policy recommended by the Board of Inquiry requires analysis, retrofit and instrumentation. The Board recommended that the Director of the Department of Transportation "Institute independent seismic safety reviews for important structures;" "...Perform comprehensive earthquake vulnerability analyses and evaluation of important transportation structures throughout the State, including bridges, viaducts, and interchanges, using state-of-the art methods in earthquake engineering;" and "...Implement a comprehensive program of seismic instrumentation to provide measurements of the excitation and response of transportation structures during earthquakes." [1, p. 10]

For specific structures, the Board recommended for action that the Transportation Agencies and Districts "Perform comprehensive earthquake vulnerability analyses and evaluation of important transportation structures - e.g. the BART Trans-bay Tube and Golden Gate Bridge - using state-of-the-art methods in earthquake engineering and install seismic instrumentation." [1, p. 11]

One of the first objectives in implementing a state-wide policy, for both Caltrans and various Transportation Districts should be the installation of seismic instrumentation on potentially vulnerable structures. The Report emphasized that: "It is likely that several earthquakes will shake a structure before an earthquake sufficient to cause damage occurs. The installation of seismic instrumentation on these structures offers the opportunity to understand their actual performance during earthquakes and anticipate their performance in large earthquake. Such information may prove to be critical in determining whether seismic upgrading is required to ensure adequate future earthquake performance." [1, p. 87]

Instrumentation must be accompanied by rigorous seismic assessment of critical structures, such as the Golden Gate Bridge, including a program detailed by the Board in their Report [1, p. 153] as follows:

"Each seismic safety assessment should include the following components:

- "1. A seismic hazard analysis to establish the annual probability of exceedence relation for peak free-field ground acceleration on firm soil and/or bedrock at the bridge site.
- "2. Analyses of the vertical and horizontal free-field ground motions, as a function of soil depth, at each girder-support location under both moderate and maximum credible earthquake conditions. New borings, using currently acceptable standards, should be drilled to bedrock at selected pier locations.
- "3. Three-dimensional dynamic analysis of the complete bridge system (superstructure and substructure), including soil-structure interaction, when subjected to the free-field ground motions at all support locations under both moderate and maximum credible earthquake conditions.
- "4. Assessment of the performance (linear and nonlinear) and safety of each structural element in both the superstructure and substructure systems under both moderate and maximum credible earthquake conditions.
- "5. Identify the deficiencies on all bridge systems that have the potential to cause unsatisfactory performance under moderate and/or maximum credible earthquake conditions.
- "6. Develop retrofit measures that would remove the deficiencies and give reasonable assurance of satisfactory performance."

The Board estimated that implementation of the policies detailed in their report would take approximately ten years for completion.

These recommendations of the Governor's 1989 Board of Inquiry are considered a minimum response for consideration by the Golden Gate Bridge, Highway and Transportation District in determining what action must be taken to assure the continued operation of the bridge following a major earthquake in the San Francisco Bay area. These guidelines formed a framework for the seismic analysis, evaluation and recommendations of this report. The seismic evaluation reported in the following chapters is a first step in implementing the complete scope of work recommended by the Governor's Board of Inquiry.

3. GROUND MOTIONS

Ground motion studies were performed to determine the seismic input to be used for the structural analysis of the bridge. Ground motions are characterized by peak ground motion parameters (accelerations, velocities, and displacements), by response spectra, and by time histories of acceleration, velocity, and displacement in the free field and at the various supports of the structure.

Determination of ground motion for the seismic evaluation of the Golden Gate Bridge was based on evaluations of the geology and topography of the area surrounding the bridge site, seismic risk analyses, and computer simulations of fault ruptures on nearby faults.

3.1 REGIONAL GEOLOGY

The Golden Gate Bridge is located in the Coast Ranges geomorphic province of central California. The Coast Ranges are characterized by northwest-southeast trending ridges with intervening valleys. The Golden Gate is the only ocean outlet for the fresh waters that flow from the north and east into the San Francisco Bay. During glacial periods, the last one occurring about 18,000 years ago, eustatic sea level was lowered as much as 300 feet and the rivers that now outlet into the northern and eastern reaches of the Bay flowed in channels through the Golden Gate onto a coastal shelf area. Most of the topographic relief of the Gate is the result of these rivers downcutting through the rocks of the now-divided San Francisco Peninsula and the southern tip of Marin County.

The ridges and intervening valleys of the Coast Ranges are controlled by faults and folds in the Franciscan Complex bedrock. The Franciscan is composed of sandstones, shales, greenstones, cherts, and melange, with the sandstones and shales often interbedded. The greenstones underlay the chert sequences. Ultramafic rocks, chiefly serpentine, though not considered to be a part of the Franciscan complex, are usually found in association with Franciscan bedrock units and commonly follow similar structural trends. The Ultramafic rocks were emplaced at low temperatures and therefore the contacts are faults rather than high temperature intrusive emplacements. The bedrock is locally overlain by soils composed of colluvium, alluvium and landslide deposits. These deposits are found along the Marin approaches to the bridge.

A geologic cross-section of the Golden Gate Bridge area is shown in Fig. 3.1. This cross-section was compiled using previous cross-sections, published geologic mapping of the area, and other unpublished exploration data provided by the Golden Gate Bridge, Highway and Transportation District, including but

not limited to the original rock corings for the Marin and San Francisco anchorages and piers. This cross-section differs from those used in the original design of the bridge [1] because of the findings of the more recent studies. As shown in Fig. 3.1, the materials underlying the Golden Gate include (1) sandstone; (2) serpentine; (3) chert; and (4) greenstone. Surficial soils are not shown on the cross-section because they are not considered significant at this scale.

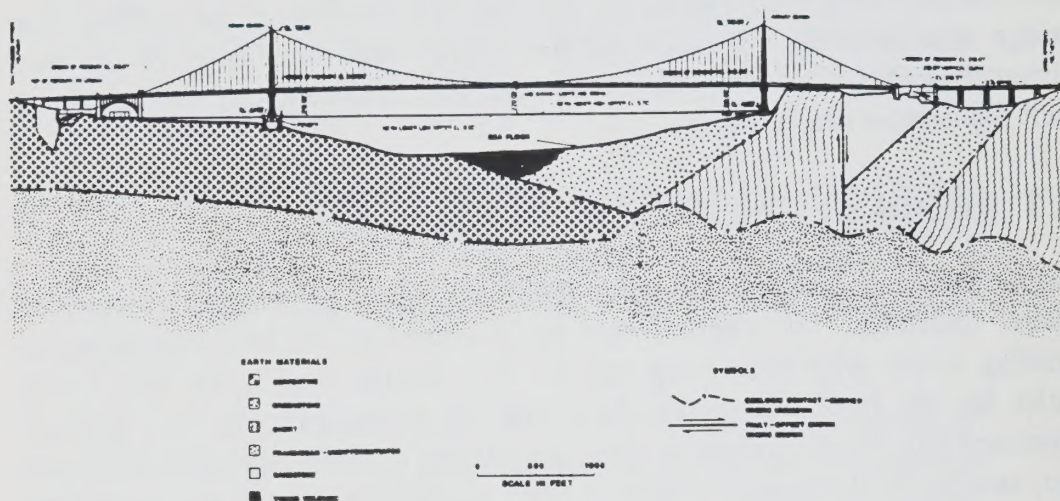


Fig. 3.1: Geologic Cross Section

3.2 GEOLOGICAL AND GEOTECHNICAL STUDIES

Initial explorations for the anchorage and pier areas of the San Francisco and Marin approaches of the Golden Gate Bridge were completed in 1931 by the Bridge District. The subsurface explorations were supervised by consulting geologist Andrew Lawson and consisted of a total of 30 corings; 17 taken in December 1930 and 13 taken in 1931. The deepest of these corings for each of the construction areas are summarized in Table 3.1.

Table 3.1: Maximum Rock Coring Depths, 1930 - 1931

Location	Depth of Coring (ft) (below lower low water)
San Francisco anchorage	-89
San Francisco pier	-191
Marin pier	-111
Marin anchorage	-3

More recent geotechnical investigations were performed on the Marin approach by Dames and Moore in 1960 and Woodward-Clyde Consultants in 1979. These investigations were for the purpose of widening the roadway and for proposed strut anchors to be installed on the roadway approach supports.

Many sources of geological information were used in this study. Two particularly valuable sources were the 1972 "Earthquake Safety Evaluation" of the bridge site, comprised of reports from three consultants, and a mapping entitled "Geology of the North San Francisco Quadrangle," by Schlocker and others in 1958.

The estimates of seismic velocities of the materials underlying the Golden Gate were gathered from numerous sources. These included (a) compilations of results of many seismic reflection surveys of the central California Coastal Ranges by Fuis and Mooney of the U.S. Geological Survey (paper presently in preparation), (b) published laboratory measurements of velocities of similar rock types, and (c) discussions with individuals having access to seismic velocity measurements for projects with similar rock formations. Based on evaluation of all the available information and some reasonable estimates of the degree of weathering of materials underlying the Golden Gate, a uniform shear wave velocity of 4000 ft per second was considered reasonable for the analyses.

3.3 FAULTS AND FAULT ACTIVITY IN THE AREA

The faults most critical to the seismic risk at the Golden Gate Bridge are the San Andreas fault, the Hayward fault, and the Calaveras fault. These faults are illustrated in Fig. 3.2; their properties are summarized in Table 3.2.

3.3.1 The San Andreas Fault

The San Andreas fault is perhaps the most studied fault in California. The fault produces right-lateral offsets and extends from Cape Mendocino to the Salton Sea, a distance of about 1000 km. Seismological, geodetic and geological studies have concluded that the seismic behavior of the fault varies markedly along the strike. Therefore, the San Andreas may be considered as a number of segments with each segment characterized by a different mode of seismic behavior.

The Holocene and historical slip rate of the San Andreas decreases northward from 32 mm/year near Bitterwater to about 12 mm/year along the San Francisco Peninsula. The decrease in the slip rate reflects a partitioning of slip to the Calaveras-Hayward fault system. Geodetic observations show that at the latitude of the southern San Francisco Bay, the cumulative rate of relative motion across the San Andreas, Hayward and Calaveras faults is equivalent to about 32

mm/year of right lateral displacement. An average slip rate of 25 mm/year was assumed for the seismic exposure studies performed during this investigation.

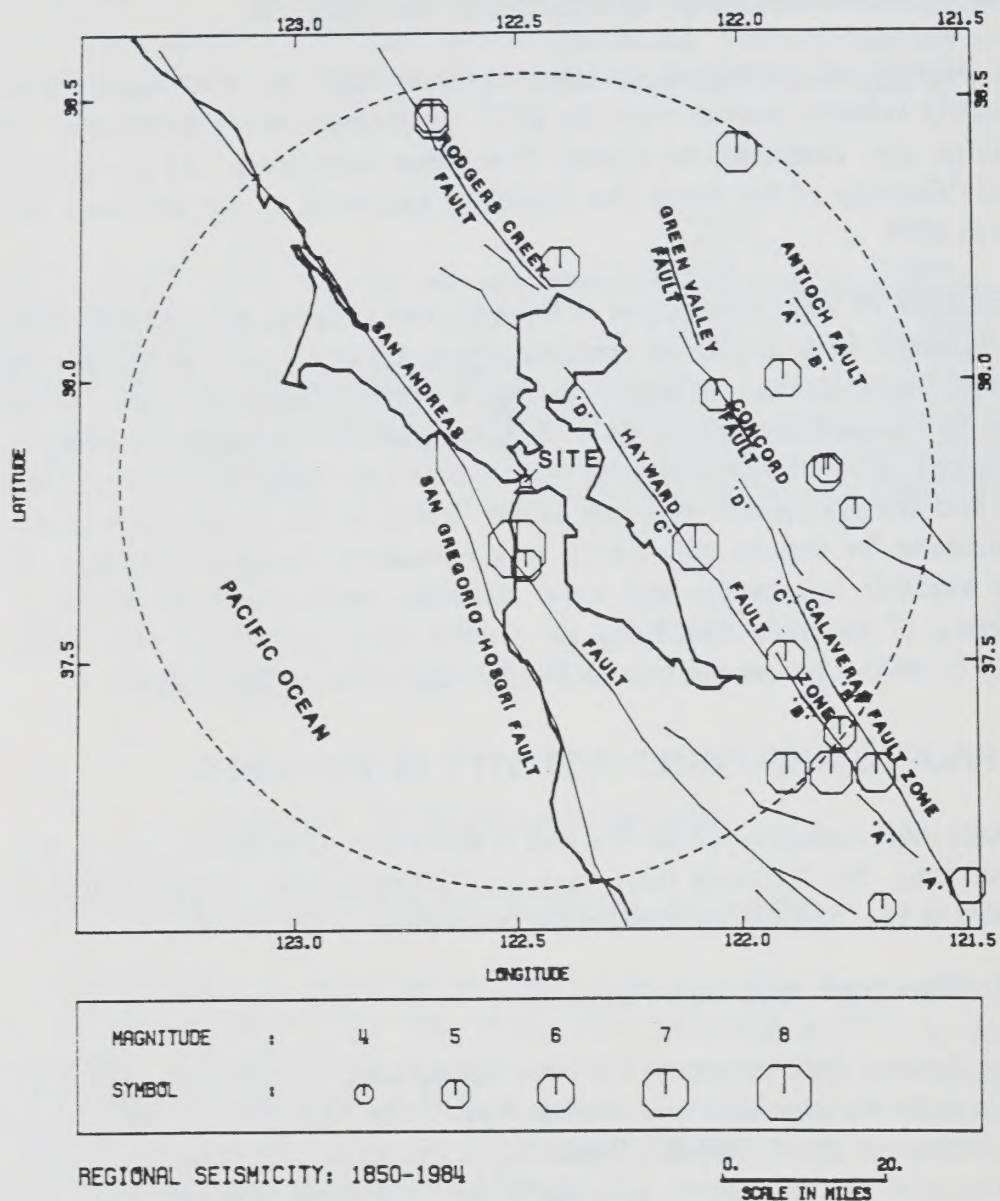


Fig. 3.2: Regional Seismicity

3.3.2 The Hayward Fault

Second in proximity to the Golden Gate, the Hayward fault splays off the Calaveras fault and strikes northwestward, a distance of about 120 km. Motion across the fault is predominantly right lateral, except for the southern part, where the fault trace is more complex and characterized by reverse

displacement. Based on the locations of historical earthquake ruptures, mapped discontinuities along the fault strike and the sense of displacement registered across the fault, the Hayward fault zone can be divided into the four segments shown in Fig. 3.2. Segments A and B include the Silver Creek-Coyote Creek and Evergreen fault systems, respectively. They show predominantly reverse type motion and are about 18-25 km long. The 50-km segment of the Hayward fault that probably ruptured in the magnitude 6.8 earthquake of October 21, 1868, is labeled as Segment C. Segment D is the remaining northernmost extension of the Hayward fault zone. The Hayward fault was assumed to have a slip rate of 7 ± 2 cm/yr.

Table 3.2 Maximum Credible Earthquakes On Significant Faults

Fault	Closest Distance to Site (km)	Fault Length (km)	Magnitude of Maximum Earthquake	a	b	Slip Rate (cm/year)
<i>Hayward</i>		124				
<i>Segment A</i>	72.1		6.5	0.73	0.62	(low)
<i>Segment B</i>	46.4		6.5	0.73	0.62	(low)
<i>Segment C</i>	25.8		7.3	2.09	0.62	0.4 - 0.8
<i>Segment D</i>	35.3		7.3	2.09	0.62	0.4 - 0.8
<i>Calaveras</i>		122				
<i>Segment A</i>	90.1		6.5	0.52	0.42	1.7
<i>Segment B</i>	59.1		6.5	0.52	0.42	1.7
<i>Segment C</i>	46.0		6.5	0.52	0.42	0.7
<i>Segment D</i>	41.7		6.5	0.52	0.42	0.7
<i>San Andreas</i>	4.0	880	8.25	3.32	0.70	3.0
<i>Seal Cove - San Gregorio</i>	13.5	176	7.5	2.62	0.75	0.7 - 1.9
<i>Rodgers Creek - Healdsburg</i>	60.0 91.1	112	7.0	1.50	0.62	0.7
<i>Green Valley - Concord</i>	55.9 47.6	90	6.75	2.06	0.81	0.05 - 0.2

3.3.3 The Calaveras Fault

The Calaveras fault, while most distant from the Golden Gate, is integrally linked to the Hayward and San Andreas faults. The Calaveras fault zone has been divided into four separate segments. Each segment is about 25-30 km in length and is assumed to be capable of producing moderate-sized earthquakes.

These segments, labeled A, B, C, and D in Fig. 3.2, were characterized in the following way for this study.

Segment A corresponds to the aftershock zone of the magnitude 5.9 Coyote Lake earthquake of August 6, 1979. The northern end of this segment is approximately coincident with the bifurcation of the Calaveras and Hayward fault zones. The southern end is located at the fault intersection with the small left lateral Busch fault, site of the magnitude 5.1 Thanksgiving Day earthquake of 1974. This segment was assumed to produce earthquakes of moment magnitude 5.9 every 82 years.

Segment B extends the length of the aftershock zone of the magnitude 6.1 Morgan Hill earthquake of April 24, 1984. The lateral extent of rupture during this earthquake appears to be controlled by mapped complexities in the fault trace. This segment was assumed to produce earthquakes of moment magnitude 6.3 every 150 years.

Segments C and D correspond to a 55 km extension of the Calaveras fault northward of Segment B. A moderate event in 1961 produced 8 km of surface rupture along the northern end of the Calaveras fault. For risk analysis, this portion of the fault was divided into two segments, C and D. Segment C strikes north from Segment B to a relatively sharp left bend in the fault that, in turn, coincides approximately with the point where the Verona fault strikes into the Calaveras. Segments C and D are considered to be similar to segments A and B with respect to seismic risk to the Golden Gate Bridge.

3.4 SEISMIC RISK

The maximum credible earthquake values assigned to the various faults and their segments as used in this study are summarized in Table 3.2. These maximum magnitudes are based on relationships between the length of surface fault rupture and the earthquake magnitude developed by Bonilla [12], Albee and Smith [13], Wyss [14], and Slemmons and Chung [15].

Moderate earthquakes have occurred in the region that are not related to known faults. To incorporate the risk from these random sources, a background source, i.e. a random event that might occur anywhere within the source region, with a magnitude of 6 was used in the analysis. Twenty percent of the total seismic activity in the region was assigned to the background.

3.4.1 Earthquake Recurrence Relationships

The earthquake probabilities for the faults and their segments were developed using a magnitude-frequency relationship derived from the seismicity catalogs and the fault activity based on their slip rates. Earthquake recurrence was modeled using a truncated form of the Gutenberg-Richter magnitude-frequency relation given by

$$\log N = a - bM$$

where N is the cumulative number of earthquakes with magnitude greater than or equal to M .

The a - and b - values were estimated for each seismic source. All aftershock sequences were removed from the earthquake catalog because statistically the mainshock is not a member of the aftershock population. The resulting a - and b -values are listed in Table 3.2, as are the estimated slip rates used to define the activity of these faults.

3.4.2 Ground Motion Attenuation

For study of the ground motion attenuation, the model of Campbell [16] was used. For this model, the peak ground acceleration is dependent on earthquake magnitude and on distance from the site. The peak ground accelerations (PGAs) were assumed to be log-normally distributed about the mean, with a constant standard error of 0.374 on a natural logarithm of PGAs for all magnitudes and distances.

3.4.3 Probabilities of Exceedence

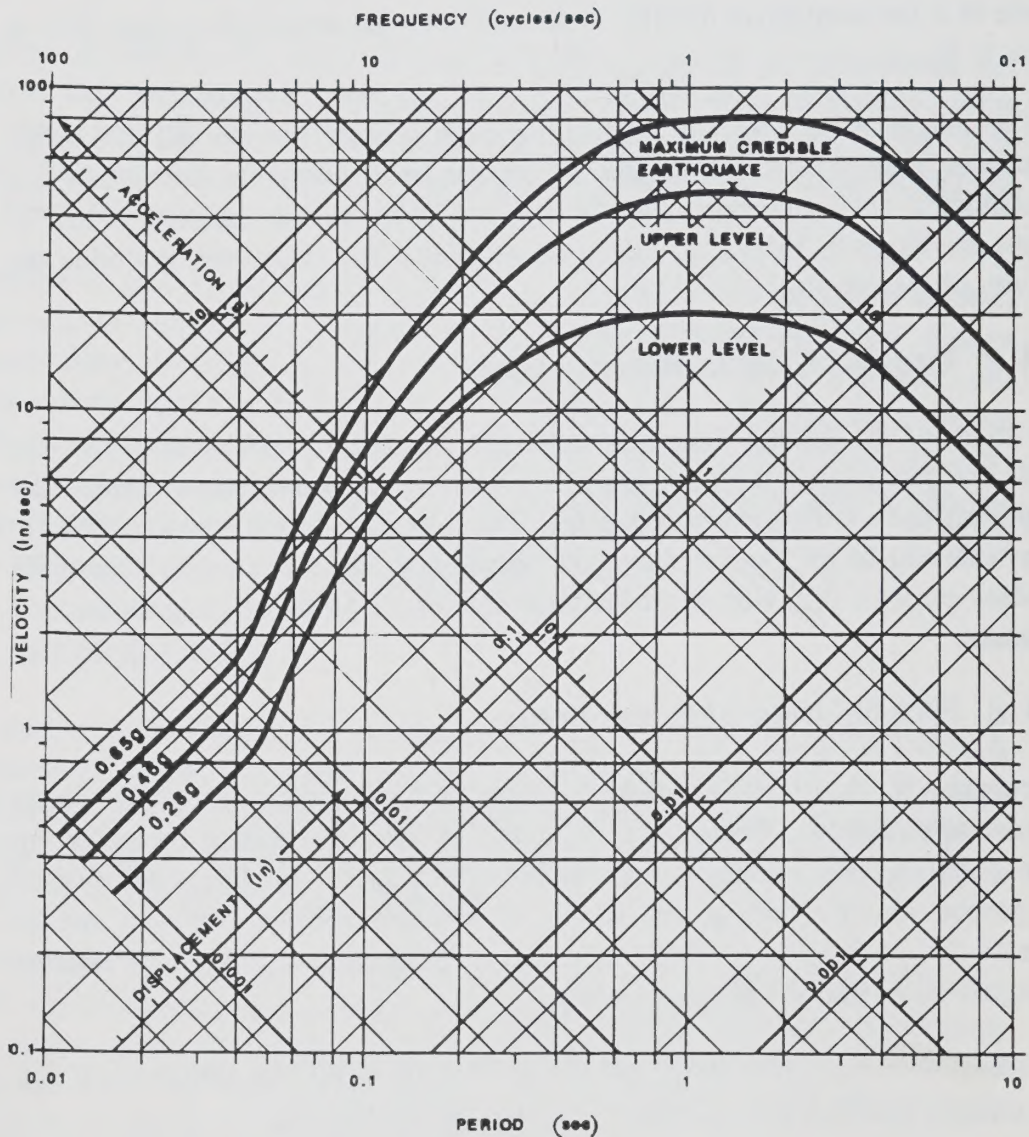
Seismic risk for the site was evaluated using a probabilistic assessment of ground motions as outlined by Cornell [17]. This method involves using a model for the seismic activity of pertinent seismic sources and a model for the attenuation of the ground motion from source to site to evaluate seismic exposure and to estimate the probability of the exceedence of peak ground motion parameters during the estimated life of the structure.

The probability of exceedence $P_E(Z)$ at a given level of ground motion, Z , at the site within a specified time period, T , is given by the formula:

$$P_E(Z) = 1 - e^{-\lambda(Z)T}$$

where $\vartheta(Z)$ is the mean annual rate of exceedence of ground motion level Z . Different probabilities of exceedence may be selected, depending on the level of performance required.

The results of the seismic risk analysis indicate that the peak ground acceleration with a 50- and 10- percent chance of being exceeded in a 50 year period at the site corresponds to 0.28 and 0.46, respectively. The peak ground acceleration estimated for a maximum credible event comparable to the 1906 San Francisco earthquake is on the order of 0.65g.



LOWER LEVEL: 50 PERCENT CHANCE OF EXCEEDENCE IN 50 YEARS
UPPER LEVEL: 10 PERCENT CHANCE OF EXCEEDENCE IN 50 YEARS

Fig. 3.3: Elastic Response Spectra for 5% Damping

3.5 DESIGN RESPONSE SPECTRA

The shape of the site-specific response spectra is influenced by the source as well as the soil conditions at the site [18]. Therefore equal probability spectral shapes were developed, with the site classified as a rock site. The recommended elastic site specific response spectra for the 50-percent chance and 10-percent chance of being exceeded in a 50-year life at the site for 5 percent damping are presented in Fig. 3.3, along with the spectra for a maximum credible event comparable to the 1906 San Francisco earthquake.

3.6 GROUND MOTION TIME HISTORIES

No records are available of strong ground motions from earthquakes occurring in similar seismotectonics and equal in size the 1906 San Francisco earthquake at close-in distances. Therefore, strong motions were generated synthetically in the frequency domain [19], allowing specification of both amplitude and phase spectra.

The effects of source, travel path and local soil conditions are contained in the Fourier Amplitude and Fourier Phase spectra in the generation of time histories. The Fourier amplitude spectra were modeled using the method suggested by Boore [20] and Brune [21], a far field model in that it assumes that the slip occurs simultaneously along the entire fault, corresponding to an infinite rupture velocity. Although this model adequately models the frequencies about 1 Hz, it does not model the long period part of the spectrum to include effects of source directivity, which were modelled analytically using a Haskell type dislocation and constant rupture velocity suggested by Aki and Richards [22].

The phase spectra model the characteristics of the strong motion accelerograms, and are therefore more complex. The characteristics of the ground motion can be preserved using either recorded phase spectra or synthetic phase spectra based on the phase properties of recorded motions. For this study, synthetic phase spectra were used that allow proper wave types in the time histories.

The synthetic time histories were developed as control motion using the maximum credible earthquake response spectra (Fig. 3.3) as a target. The transverse control motion time history is presented on Fig. 3.4, its response spectra are presented in Fig. 3.5, They are reflective of a maximum credible earthquake similar to the 1906 San Francisco earthquake occurring at a distance of approximately 10 km from the Golden Gate Bridge site.

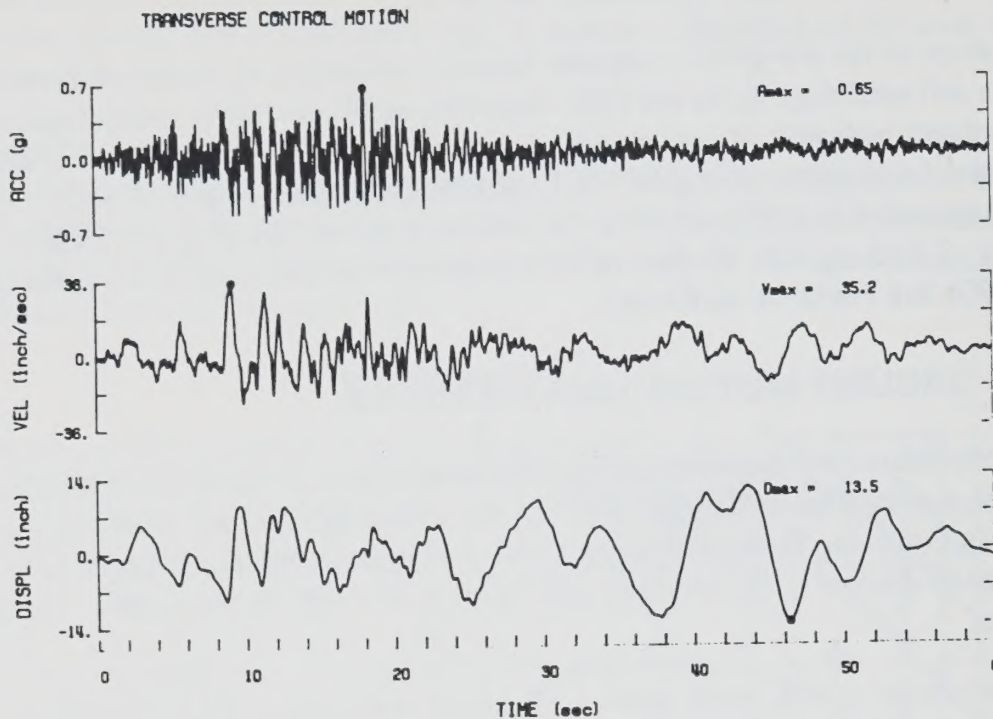


Fig. 3.4: Transverse Control Motion Time History

3.7 MULTIPLE-SUPPORT TIME HISTORIES

Time histories for multiple support excitation must contain proper estimates of the phase delays that are reflective of both travelling waves and the velocity of the media in which they travel. The histories must also contain proper coherency and incoherency that reflect the rupture scenarios and wave travel media.

In estimating the multiple support excitation, two rupture scenarios of the San Andreas Fault were considered. These represent the two most crucial cases of ground motions that most likely will bound the response of the bridge.

The first scenario consisted of 400 km of San Andreas Fault rupturing adjacent to the bridge with the bridge being located almost at the mid-point of the rupture length. This scenario will produce the highest amplitudes of the motion with the least delays in the shear wave part of the motion and a significant phase delay in the surface wave part.

The second scenario consisted of 400 km of the San Andreas Fault rupturing, but with the rupture stopping about 20 km south of the bridge. This scenario represents the case that will produce somewhat lower amplitudes of ground

motion at the site but will have considerable phase delays in motion for the shear wave as well as the surface wave part of the ground motion.

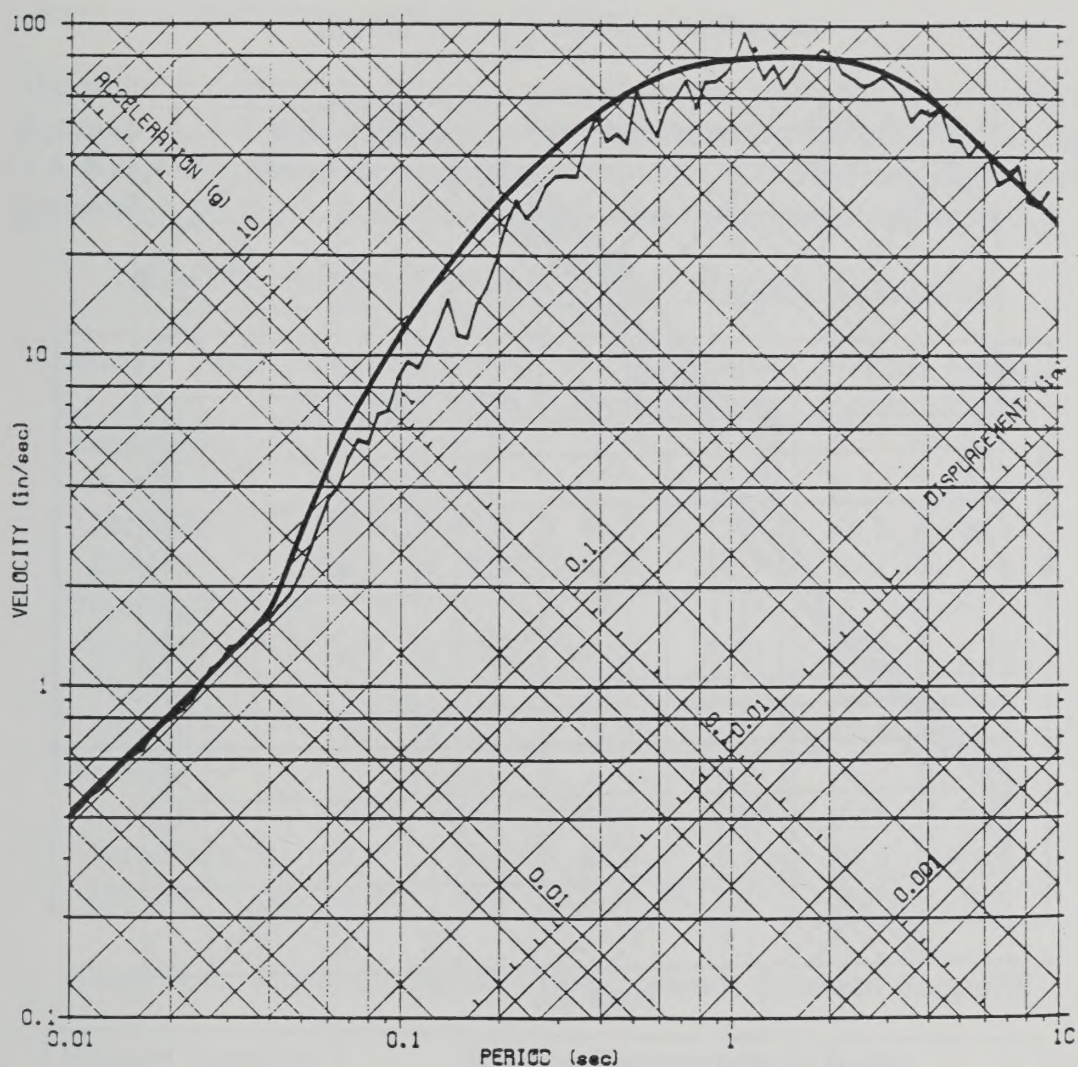


Fig. 3.5: Control Motion Spectra for 5% Damping

The first scenario was considered most crucial for the structural response, and was modeled using a three dimensional finite element soil structure interaction program (SASSI; Lysmer, Tabatabaie, et. al.) [23]. The program has the capability of handling wave propagation through layered media arriving at the site at different angles of incidence and from different azimuths.

To model the effect of the traveling wave and the incoherency and coherency of the ground motions, the San Andreas fault was segmented into four different portions, each reflecting different reasonable average estimates of wave travel path in terms of azimuths and angles of incidence at the receiving site. For these portions of the fault, the analysis employed propagation of coherent plane P,

SV, R, and SH waves. The results of these analyses were saved as transfer functions and later used in conjunction with control motion derived earlier.

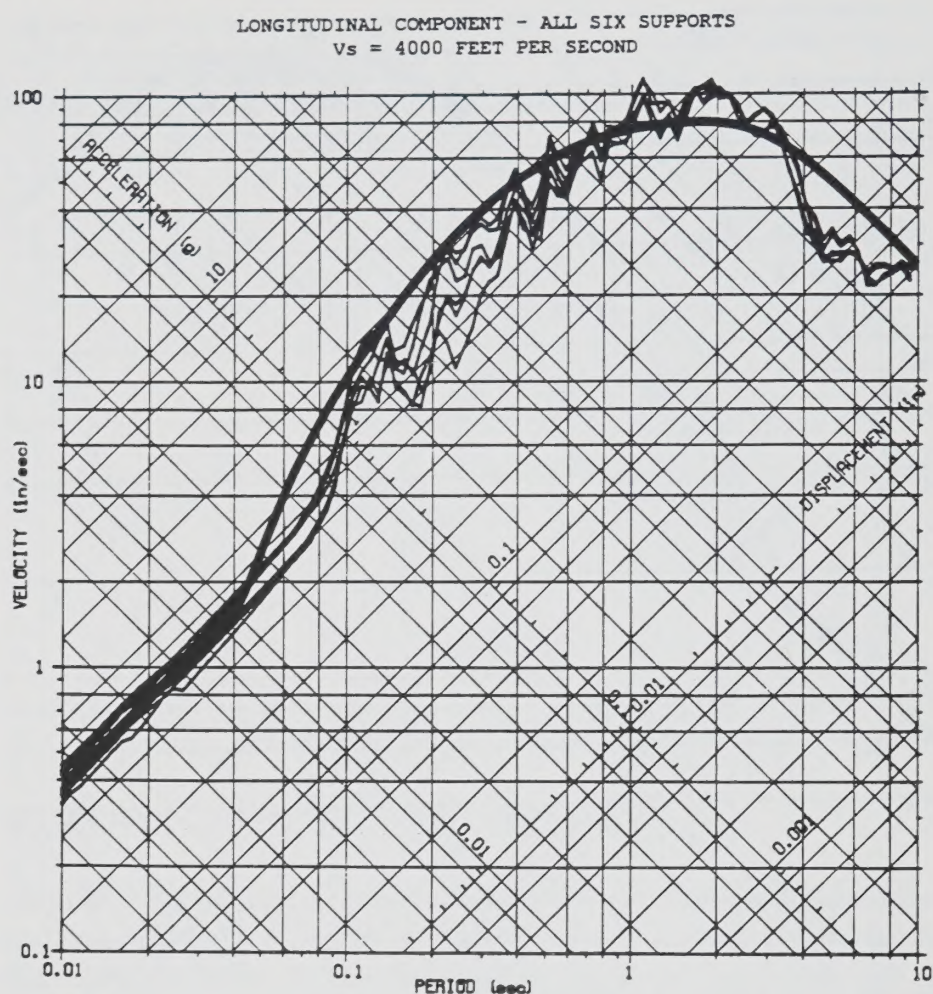


Fig. 3.6: Multiple Support Motion Spectra for 5% Damping

The synthetic motions at other supports of the bridge were generated by modifying the control motion at different support points by use of transfer functions that reflect the relative changes in motion at the other point as a result of the wave travel from different parts of the fault. This was achieved by varying the relative amounts of wave types from different parts of the fault to the P-wave, S-wave and surface wave windows of the control motion.

For the multiple-support time histories the spectra for the horizontal components show slight deviations from the target spectra (i.e. the maximum credible event) as well as variations between the different supports. The deviations in spectra reflect the properties of the wave train from real earthquakes, and the variations

between supports reflect the coherency and incoherency of the waves. For the vertical component, the spectra are about 2/3 of the target spectra. This is in accord with observations made close to seismic sources.

The multiple support time histories generated for this evaluation are included in Appendix A. Response spectra for each support motion are presented in Fig. 3.6. Summary statistics for the peak ground motion at the different supports are presented in Table 3.3. Peak relative displacements are presented in Table 3.4.

Table 3.3: Summary Of Peak Ground Motions At Supports

Support Point	North Anchor	North Pylon	North Tower	South Tower	South Pylon	South Anchor
<i>Longitudinal</i>						
Acc (g)	0.54	0.6	0.73	0.7	0.58	0.68
Vel (in/sec)	42.2	41.0	44.2	43.9	40.8	40.1
Dis (in)	12.9	12.9	13.5	14.7	15.1	14.9
<i>Transverse</i>						
Acc (g)	0.69	0.50	0.65	0.60	0.60	0.58
Vel (in/sec)	44.3	40.2	51.9	54.1	38.7	39.9
Dis (in)	17.2	17.7	18.8	18.1	17.5	17.4
<i>Vertical</i>						
Acc (g)	0.56	0.56	0.60	0.74	0.83	0.76
Vel (in/sec)	26.5	28.6	29.2	27.6	29.6	28.6
Dis (in)	10.9	10.6	9.4	7.6	9.2	9.3

Table 3.4: Maximum Relative Displacement Between Supports

Support Locations	Relative Displacement in Inches		
	Longitudinal	Transverse	Vertical
<i>North Anchor to North Pylon</i>	1.5	2.8	1.3
<i>South Anchor to South Pylon</i>	0.7	12.1	0.6
<i>North Anchor to North Tower</i>	3.9	7.0	3.7
<i>South Anchor to South Tower</i>	3.6	5.3	3.8
<i>North Tower to South Tower</i>	19.7	5.5	4.6

4. SEISMIC PERFORMANCE CRITERIA

The need for retrofitting an existing bridge to withstand a future earthquake is determined by evaluating the structure against performance criteria specifically developed for the bridge. Any component that meets the specified criteria does not need retrofit; any component that does not meet the criteria needs retrofit. Gross non-compliance with the criteria would suggest a need for a system retrofit in addition to a component retrofit. The establishment of the performance criteria involves not only evaluation of engineering issues regarding the structure but also public policy issues, economic issues, and various other legal, social and economic considerations regarding the operation of the bridge.

4.1 POLICY ISSUES

The general performance criteria for the seismic evaluation of the Golden Gate Bridge were derived from those discussed in the *Report of the Governor's Board of Inquiry* [1]. These criteria specify the degree of performance expected of the structure following any given size earthquake. It should be stressed that the magnitude of earthquake for which the bridge must remain functional is a policy issue rather than an engineering issue. Modern engineering technology can be used to retrofit the Golden Gate Bridge so it can withstand essentially any magnitude of earthquake; the primary limitations are time and money. Recognizing this, consulting engineers can make recommendations on the magnitude of earthquake which the bridge should survive, but the District must set policy.

The level of performance expected of the bridge used as a basis for this seismic evaluation addresses three different magnitudes of earthquake:

- a. *For frequent but small earthquakes:* Permanent damage to structural elements should be minimal and confined to small areas of non-essential components. Immediate repair of the damage should not be required and the damage should have no effect on margins of safety that existed before the earthquake.
- b. *For moderate earthquakes:* Some small permanent damage and residual stresses and displacements can be tolerated as long as damage is in non-critical elements, does not disrupt regular function of the bridge, and can easily be repaired if necessary.
- c. *For maximum credible earthquake:* The bridge should remain open to traffic. The extent of tolerable damage and degree of usefulness of the bridge after a major earthquake is a policy issue that should be determined by the District.

Some options for condition c which may be evaluated by the District in the determination of its policy, include determination of whether:

1. All traffic lanes should remain open and functional.
2. A number of lanes (to be specified) should remain open for cars and trucks. The closed lanes should be opened in a few days following the event without major repair and retrofitting efforts.
3. A number of lanes (to be specified) should remain open for cars only. The bridge should be open to regular car and truck traffic in a few weeks without major repair or retrofit.
4. Two lanes remain open for emergency vehicles only. The bridge should return to its normal function in a few weeks with reasonable but not substantial repair efforts and costs.

In any of the above cases, the bridge should not be closed for more than one day. The lane closure or lane load reductions should not last more than two weeks. Temporary or intermittent closure of curb lanes for the inspections which should be carried out after any major earthquake should be expected.

The durations of lane closures are somewhat arbitrary; more rational performance criteria can be established by considering the economic impact of the criteria.

Final policy choices on these issues must be made by the District.

4.2 ENGINEERING ISSUES

Once the level of performance required to satisfy external social, political and economic requirements has been established, the engineering issues involve the technology by which the bridge structure is to be evaluated.

No current bridge design specification, including *AASHTO Specification* [24] and the *Caltrans Bridge Design Specification* [25], sets explicit standards for either the seismic retrofit of bridges or for the repair and seismic upgrading of damaged structures. While the general provisions of these design specifications were used in this evaluation in instances where applicable, it was necessary in this evaluation to develop additional project-specific standards which address the particular problems in this historic structure. Since some components of the bridge are better addressed by current building standards, some building-specific references as well as bridge-specific references were used. The standards derived

for this investigation were based on the AASHTO specifications and Caltrans specifications for design of bridges, the *AISC/LRFD Specification* [26], the draft *AISC/LRFD Seismic Specification* [27], the *Uniform Building Code* (UBC) [28], and the *Guide to Stability Design Criteria for Metal Structures* [29].

The seismic performance of the bridge structures was assessed from the perspectives of comparing performance demand with performance capacity, which are defined as follows:

1. *Demand*: Maximum forces and deformations caused by earthquakes ranging from more frequent moderate events to the maximum credible event. The seismic demands on the bridge are determined by analytical modeling and response analysis and are discussed in Chapter 5 (for the suspension bridge) and Chapter 6 (for the approach structures).
2. *Capacity*: Maximum force and deformation levels that the structure and its components can tolerate. The seismic capacities of the bridge system and its components are discussed in this chapter.

Knowledge bases of strong ground motion in the long period range and spatial variability of ground motion at rock site are both limited at the present time. This uncertainty in strong motion seismology results in some uncertainty in the performance demands on the structures and their components. Given the uncertainty in demand, the capacity aspect of the evaluation is crucial to provide that enough strength, ductility, redundancy, and deformation capability are built into the structural systems.

4.3 ACCEPTANCE CRITERIA

The following discussion applies to both the approaches and the main bridge. The structures include steel truss systems with large numbers of axially loaded members and gusset plate connections, steel plate girders, the cables, suspenders and cellular towers of the main bridge, and the reinforced concrete structures of the anchorages and their housings, as well as the pylons and piers.

The basic loading combination under which the structures were evaluated consisted of dead load and earthquake effects, in compliance with the Caltrans requirements [25]. Under this combination, the basic strength criteria for members was set using a load factor of 1.0. To achieve the goal of the bridge remaining fully functional following a major earthquake, at least the following acceptance criteria should be met.

The criteria listed here are intended for evaluation of the structures and to provide a preliminary basis for the design of retrofit for existing components. In actual design of retrofit measures, more development will be required, including refinement of these criteria and evaluations of desirable upgrades to the seismic resisting systems as well as their components.

4.3.1 Truss Capacity Criteria

Those members in the approach viaducts and in the suspension bridge which resist load primarily by direct axial stress but which are also subject to "secondary" bending stresses are classified as truss members. These truss members are further classified as (a) main members with no redundant load path, (b) main members with redundancy, and (c) secondary members and non load bearing bracings. Joints between truss members, structural bearings, and foundation connections in the approach viaducts are also considered here and bear the same classification as the members they connect. The performance criteria are more restrictive for the lower-numbered categories, due to the more dire consequences of their unsatisfactory performance.

The following criteria were adopted for these sub-classifications:

a. Main Members with no Redundant Load Path

1. The members shall not develop overall buckling. The AISC-LRFD Specification shall be used to establish the buckling capacity. The KL/r of these members shall not be more than $720/\sqrt{F_y}$. This is the limit that is imposed on bracing members subjected to axial load in the current AISC-LRFD Seismic Code [27].
2. The members shall not develop local buckling. Satisfy provisions in Table EQ7-1 of [27]. This means that for unstiffened elements of a cross section, b/t shall be less than $52/\sqrt{F_y}$. For stiffened elements the following formulae shall apply:

$$\text{For } \frac{P_u}{\Phi_b P_y} < 0.125, \quad \frac{b}{t} < \frac{520}{\sqrt{F_y}} \left[1 - \frac{1.54 P_u}{\Phi_b P_y} \right]$$

$$\text{For } \frac{P_u}{\Phi_b P_y} > 0.125, \quad \frac{b}{t} < \frac{190}{\sqrt{F_y}} \left[2.33 - \frac{1.54 P_u}{\Phi_b P_y} \right] > \frac{253}{\sqrt{F_y}}$$

where Φ_b is 0.9, P_u is the actual applied axial load and P_y is the axial yield capacity of the member. F_y is the yield stress of the material.

3. The major axial load carrying members with no redundancy shall not yield. Use AISC-LRFD Specification [26] to check the condition:

$$P_u < 0.9 A_g F_y$$

4. No fracture of net section shall be permitted since this can produce a brittle failure. Use AISC-LRFD Seismic Code [27] or UBC-88 [28]:

$$\frac{A_e}{A_g} > \frac{1.2 \alpha P_u^*}{\Phi_t P_n}$$

5. Check buckling of individual elements of built-up members. L/r of individual elements shall be less than $0.75 K l/r$ of the member [28].

b. Main Members with Redundancy

1. The members shall not develop severe overall buckling. Minor buckling may be permitted. A ductility demand of at least 3 may be allowed. In this case, axial load carrying capacity shall be modified for second cycles and beyond, UBC, p.569 [28].

$$F_{as} = \frac{F_a}{\left[1 + \frac{KL/r}{2C_c} \right]}$$

2. The members shall not develop severe local buckling. Provisions in Table B5.1 of AISC-LRFD Specification [26] for λ_p shall be satisfied.
3. These members may yield in tension.
4. No fracture of net section is permitted since this can produce a brittle failure. Use AISC-LRFD Seismic Code [27] or UBC-88 [28]:

$$\frac{A_e}{A_g} > \frac{1.2 \alpha P_u^*}{\Phi_t P_n}$$

5. Individual elements of built-up members shall be checked for buckling. L/r of individual element $< K l/r$ of the member.

c. Secondary Members and Non Load Bearing Bracings

1. Minor elastic buckling may be permitted. $K l/r < 200$.
2. Members shall remain elastic to provide continuous bracing stiffness.

3. No fracture of net section shall be permitted since this can produce a brittle failure. Use AISC-LRFD Seismic Code [27] or UBC-88 [28]:

$$\frac{A_e}{A_g} > \frac{1.2 \alpha P_u^*}{\Phi_t P_n}$$

4. Individual elements of built-up members shall be checked for buckling. L/r of individual element $< Kl/r$ of the member.

d. Truss Connections

1. Connections of main truss members with no redundancy shall be equal to or stronger than the member strength of $A_g F_y$.
2. Connections of main truss members with redundant load path shall be capable of transferring the smaller value of the following:
 - a. The axial tension and compressive strength of the member.
 - b. The calculated force due to load combinations.
 - c. The maximum force that can be transferred to the truss member by the system.

However, truss connections shall be capable of transferring at least 50% of member strengths.

3. Connections of secondary members and bracings shall provide for transfer of their calculated force or 75% of their strength whichever is greater.

e. Truss Roller and Rocker Supports

Special attention shall be paid to evaluation of main truss supports, particularly anchor bolts of rocker pin supports which can be vulnerable. Overturning and slipout of rocker and roller supports shall be checked.

f. Column Base Connections

Tower base connections shall develop strength demand of the applied load in a ductile manner. Yielding of steel plates and other elements may be permitted but failure of anchor bolts, grout and concrete is unacceptable.

4.3.2 Tower Capacity Criteria

The primary components of the towers of the main suspension bridge are the steel plates which make up their cellular cross section. These plates are of carbon steel with higher strength silicon steel used in regions of high stress. They are connected at the vertical edges of the cells by riveting to rolled angles, and at horizontal joints, located at about 50 ft intervals over the height of the towers, by riveted splice plates.

Several important issues must be addressed in adopting acceptance criteria for the tower components:

1. The steel making up the towers is in itself ductile, but the connections between the plates, made with rivets and subject to tension and shear, are potentially very brittle. The failure mode in compression is buckling, which is also potentially non-ductile.
2. In compression, the probable failure mode of the plates is by buckling. Depending on the duration of the buckling-level stresses in the plates, such buckling may not adversely effect the ability of the tower to weather the earthquake, even though buckling is not inherently a ductile mode of failure. However, such buckling may be extremely difficult to repair, and such repair would be necessary in order to provide for an additional service life of the bridge.
3. In tension, the probable failure mode of the plates is by fracture of the rivets in the horizontal splices, which were designed to resist one half the yield capacity of the joined plates. This significantly limits the strength and ductility of the towers under large bending moments.
4. In shear, the plates themselves as well as the rivets joining the plates at the corners of the cells may be critical. These components are subject to internal shear stresses under uplift conditions as well as due to the normal lateral loads. Also, the shear capacity of the Tower/Pier connection under uplift may be critical. Current seismic criteria for bridges [25] require that vertical elements provide shear strength greater than that required to develop ultimate moments. While this may not be possible to assure for the Golden Gate Bridge, the limitation on seismic resistance caused by shear in plates, rivets, and the Tower/Pier connection must be checked.
5. The integrity of the towers is dependent on their stability against buckling as well as the strengths of their components. Loss of stiffness due to failure modes listed above could result in a decrease in the factor of safety against

overall buckling of the towers. Assessment of acceptable levels of damage must take this into account.

6. The reinforced concrete piers supporting the towers are also critical to tower performance. Critical issues include contact stresses, spalling, bursting, and the need for ductility. These are considered part of the reinforced concrete acceptance criteria which are discussed subsequently.

With these issues as background, the following acceptance criteria were adopted for the tower components:

1. *Tension in plates:* Failure is assumed to be by fracture of rivets at splices. The total stress shall be not greater than $\frac{1}{2} F_y$.
2. *Compression in plates:* Failure is assumed to be by yield or buckling of the plates. The total stress shall be not greater than the calculated buckling stress or F_y , whichever is less.
3. *Shear:* Shear capacity shall be provided both in the component plates and in the riveted connections.
4. *Riveted connections:* The total nominal stress in a rivet shall not exceed its yield stress F_y .

4.3.3 Cable Capacity Criteria

Cables provide a primary component of the structural system of the suspension spans of the Golden Gate Bridge. Three-foot diameter parallel-wire cables provide the primary support to the roadway, which is hung from the cables by multiple, 2 11/16 inch diameter helical-lay suspenders. Tie-downs in the pylons, consisting of cables similar to the suspenders, maintain the geometry of the main cables and are necessary to the structural integrity of the towers. Eyebars anchor the main cables into the concrete gravity blocks, and although they are not technically cables they are also addressed here.

Acceptance criteria for these cable components are set as follows in recognition of the risk to the bridge of failure of the component.

1. *Main cables:* Failure of a main cable would result in collapse of the bridge. Damage to a main cable would be extremely costly and difficult, if not impossible to repair. Therefore the conservative criterion that total calculated stress shall be not greater than $\frac{1}{2} F_y$ is adopted for this evaluation.

2. *Tie downs:* Failure of a tie down would result in extreme geometric dislocation of the side span of the bridge and damage to the towers as a result of the change in geometry of the cable. Therefore the conservative criterion adopted for this evaluation is that total calculated stress shall be not greater than $\frac{1}{2} F_y$.
3. *Suspenders:* Failure of a suspender does not present as significant a risk to the bridge as would the failure of the other cable components listed above. Therefore the criterion that total calculated force shall be not greater than the yield force of the suspender assembly is adopted for this evaluation.
4. *Eyebars:* The number of eyebars classifies this group as a redundant system. However, a failure of an eyebar would reduce the efficiency of the main cable, would cause stress concentrations in the main cable, and would be extremely costly if not impossible to repair. Therefore, the conservative criterion adopted for this evaluation is that total calculated stress shall be not greater than $0.8 F_y$.

4.3.4 Reinforced Concrete Capacity Criteria

Reinforced concrete construction is used in the piers, anchorages, and pylons of the suspension bridge as well as in the anchorage housings and foundations of the approach structures. The reinforced concrete technology used in these structures is typical of that of the 1930's:

1. The design concrete strength was low by today's standards.
2. The reinforcing bars were of the era's "low-deformation" variety which are not capable of developing the bond strength with the concrete that is typical of today's "high deformation" bars.
3. The detailing of the reinforcing bars does not address the developments in concrete technology of the intervening years which requires more stringent embedment.
4. No confinement reinforcement is provided, which is essential for provision of the ductility essential for seismic resistance in reinforced concrete structures.

These aspects of the reinforced concrete components of the Golden Gate Bridge suggest that no modern seismic criteria for reinforced concrete structures are fully applicable to their evaluation. However, such modern criteria can provide a basis for their evaluation.

As a framework within which to evaluate these structures, the design criteria of AASHTO [24], the Caltrans Bridge Design Specifications [25], the Uniform Building Code [28], and the American Concrete Institute Building Code [30] were adopted. Strength capacities were calculated using these documents as a basis; however, no ductility was allowed due to the detailing of the structures. As a result, the components of these structures are required to resist the site-specific ground motion response spectra with load factors of 1.0 and resistance factors as specified by AASHTO.

5. SUSPENSION BRIDGE EVALUATION

The seismic evaluation of the main suspension bridge was conducted in phases, using the ground motions reported in Chapter 3, in order to capture the global response of the long structure subjected to different ground motion input at the various supports, as well as the local nonlinear behavior at the bases of the towers

5.1 ANALYTICAL MODELS OF THE BRIDGE

In the course of the investigation, several analytical models were developed, each to achieve a specific goal:

5.1.1 Global Models

At the global level, a 3D model of the main bridge, from South the (San Francisco) anchorage to the North (Marin) anchorage, was developed. To account for the large displacement effect, geometric stiffnesses were included. Out-of-phase, ground motion inputs at each of six support points were used in a direct step-by-step time history analysis. Quasi-static responses of the structure were also studied to assess the sensitivity of the structure to multiple-support displacements. Rigid-base response spectrum analyses were also conducted to assess the sensitivity of the structure to vibrational movements.

The 3D model of the main bridge used in this investigation was originally developed for T.Y. Lin International's Transit Feasibility Study [7] and represents the structure in its current configuration. The 9,933 frame elements connecting 4,775 nodes of the original model were reduced by using a superelement formulation for the stiffening trusses while maintaining the same level of refinement in the behavior modeling. The total number of active degrees of freedom in the final model was in the order of 4000.

The primary characteristic of the suspension bridge is its flexibility; the large displacement which may be induced even under service loads may result in significant distortions in the bridge geometry. To account for these second-order effects on the displacement and stress responses, a geometrically nonlinear analysis was performed to establish the current dead load configuration of the structure.

Since any dynamic loads and responses may be regarded as disturbances to the current dead-load configuration, the nonlinear analysis problem was "linearized" with respect to the dead-load state of the structure. In the global earthquake

response analysis of the main bridge, nonlinear geometric effects due to cable tensions and axial compression of the tower members were taken into account by using geometric stiffness matrices. Both site-specific response spectrum analysis and direct time integration analysis using site-specific multiple-support ground motion were carried out for the linearized global model of the bridge.

5.1.2 Local Models

At the local level, more detailed assessments were made using several linear and nonlinear models which included segments of stiffening trusses and tower structures. The most important local model was the modeling of nonlinear behavior at the connection of tower base and concrete pier. This level of refinement in the analysis accounted for improved stress redistribution at the tower base and provided for more realistic response assessments.

Based on the responses obtained from the global model of the stiffening truss, local responses (e.g. stresses in the truss chord) were determined by calculating demand/capacity ratio evaluation.

For the tower structure, a detailed local model was developed which included a nonlinear filament model at the base to represent such nonlinear responses as base plate uplifting, hold down rivet shear failure, and plate compression buckling.

For long period structures, it is generally recognized that the maximum displacement response of a nonlinear yielding structure is about the same as that of the linear structure. However, stress distributions within the two systems may be quite different. Based on this premise, a better estimate of the stress distribution within the tower was estimated by imposing on the local nonlinear tower model the maximum displacement responses of the tower obtained from the linear, global analysis of the whole bridge.

In addition, the nonlinear tower model was used to assess the capability of the tower structure to tolerate relative longitudinal ground movement between the tower and the adjacent anchorage.

5.2 STRUCTURAL MODELING

Both the global model of the entire main bridge and the local model of the tower structure were idealized as assemblies of finite elements to capture the complicated geometry, stiffness and mass effects. In the global model, the updated Lagrangian formulation accounted for large displacements and large rotation effects. This implementation was necessary because the accurate

determination of the dead load state and the corresponding tangent stiffness matrix are crucial to all subsequent earthquake and response analysis.

5.2.1 Element Types

Six basic element types were used in the analytical models:

1. Cable elements were used to model main cables, suspenders, and tie-down cables. Main cables are connected by piecewise linear elements between adjacent suspender connections.
2. 3D Frame elements were used to model members in the towers, pylons and the stiffening truss system. It was assumed that cables are hinged to the tower at the top of the tower shaft.
3. Zero-Length Hinge elements were used to model connections of the stiffening truss to the towers and to the pylons. All rotational degrees of freedom were unrestrained; all transverse degrees of freedom were restrained. Longitudinal restraints were provided only at side-span to tower connections.
4. Boundary springs were used to model the flexibility of the tower base connection to the concrete pier. A comparative study was carried out to assess the effects of the massive concrete pier and rock foundation; as expected, these have practically no effect on the tower response. Boundary spring coefficients were established based on the nonlinear behavior at the tower base accounting for the shear failure of rivet connections to hold-down angles and uplifting of base plate.

The boundary spring was used at the top of the tower model to represent the longitudinal restraint provided by the main cable. The stiffness coefficients were calculated based on strain energy considerations.

5. Two-Node Superelements were developed to represent the stiffening truss and lateral bracing system except at locations adjacent to main towers and pylons. In these transition locations, all members of the truss and lateral bracing systems were included to ensure calculating the proper transfer of shears between lateral bracing and towers.
6. Uniaxial nonlinear elements with unequal tension (uplifting) and compression yield levels were used to develop the nonlinear filament model for the connection of tower base to pier.

5.2.2 Development of Stiffening Truss Superelements

To reduce the structural model to a manageable size and to verify the physical properties of the computer model, superelements were developed for each 50 ft of longitudinal truss segment spanning between adjacent suspender panel points.

The superelement nodes are located at the center of the stiffening truss box. It was assumed that the section at each panel point is essentially rigid. The two suspender nodes at each section are connected to the superelement node by master-slave transformation. The complicated 3D space truss was reduced to a series of line elements each represented by generalized stiffness matrices load vectors, and lumped masses (rotational and translational). Each superelement replaced 67 elements in the original space truss model. Still, all structurally significant features of the suspended structure were included, e.g., coupling between torsional and lateral responses.

Physical properties of an equivalent stiffening girder were extracted from the generalized stiffness matrices as shown in Table 5.1. These physical properties include axial and shear areas, torsional constant, moment of inertia, and vertical eccentricity. These extracted physical properties were compared with values calculated by Baron [10] and served as the basic benchmark of the complicated truss system.

Table 5.1: Average Properties of Stiffening Truss Superelement

Property	Main Span			Side Span	
	Center	¼ Point	At Tower	Center	At Tower
<i>Area (ft²)</i>	3.7	4.2	3.7	2.9	2.6
<i>I₃₃ (ft⁴)</i>	574	660	575	444	409
<i>I₂₂ (ft⁴)</i>	6900	8260	7000	5340	4537
<i>Torsional Constant (ft⁴)</i>	316	319	320	305	303
<i>Vertical Eccentricity e₂ (ft)</i>	≈ 0.0	0.5	1.0	≈ 0.0	1.0
<i>Shear Area A₂ (ft²)</i>	0.45	0.46	0.45	0.41	0.40
<i>Shear Area A₃ (ft²)</i>	0.37	0.38	0.53	0.35	0.43
<i>Lumped Mass (k/g)</i>	20.6			19.4	

5.2.3 Nonlinear Filament Model of Tower Base Connection

The connections between the main tower shafts and concrete piers are (1) rivet connections of the 96 hold down steel angles to the web plates of the tower shaft, and (2) 78 steel dowels to provide some positive shear connection. The

anchoring steel angles were each pre-stretched to 105 kip before being riveted to the shaft webs. The rest of the tower base is just resting on top of the pier.

Due to the high vertical load applied to the tower base, horizontal movements between tower base and pier are effectively prevented by the friction force and also by the steel angles and dowels. The main concern during earthquake is the rocking response of the tower shaft resulting in partial uplifting and stress redistribution. The load path by which the seismic lateral load is transmitted to the piers is governed by:

1. Shear strength of the rivet hold down connection;
2. Axial (buckling) strength of the tower shaft web plate.

The yield strength of the hold down steel angle is in the order of 1500 kip for each group of four angles, much higher than the shear strength, 404 kip, of the 16 double-shear rivet connection.

The buckling stress level of the stiffened web plate (42 in. wide, 7/8 in. thick) is 45.5 ksi (about 90% of the uniaxial yield stress level).

Based on these considerations, a nonlinear filament model was developed for each tower shaft, the base area was discretized into 42 tributary areas, each being characterized by a nonlinear uniaxial force-displacement relationship. On the compression side, the yield force level was defined as $F_{cr} = f_{cr} A_i$ where f_{cr} is buckling stress level at the web plate and A_i = the tributary area of the i -th filament.

It was assumed that, under the tributary area of each filament, concrete bearing failure will not occur.

On the tension side of the uniaxial curve, the tensile yield level for most filaments is zero. Even at the outer cells where steel hold down angles are located, the tensile yield level governed by the low shear strength of rivet connection is low:

$T_u = 404$ kip for 4 angles with 4 rivets each acting in double-shear (total 16)

Uplifting of the base is mostly resisted by the dead load effect, FDL, which ranges from $4.70 A_i$ to $6.85 A_i$ depending on the location of the filament.

5.2.4 Non-linear Moment/Rotation Relationship

Using the nonlinear filament model for the tower base connection, nonlinear moment-rotation relationships for the two principal axes were developed under a constraint axial load of 70,870 kip, which corresponds to the tower weight and vertical component of the cable load applied at top of tower.

Initial elastic rotational stiffnesses are $K_z = 3,300,000,000$ kip-ft/rad and $K_x = 860,000,000$ kip-ft/rad where X and Z are the longitudinal and transverse axes of the bridge. These stiffness coefficients were used as boundary springs in the global model.

In this model, when the base rotational reaches about 0.0003 rad. in either direction, riveted hold-down connection are sheared off at some locations and partially uplifting occurred. However, the average filament compressive stress reached buckling stress level (as defined earlier) only when the base rotation reached about 22×10^{-4} rad., which corresponds to a ductility factor of about 7 for the base moment. Between these two base rotation limits, partial uplifting occurred; neutral axis shifts; and stress redistributes within the base. Once uplifting occurred, the base connection becomes softened and in a elasto-plastic nonlinear analysis, the base moment will be limited. The yield moments corresponding to initial rivet shear failure are $M_{zy} = 900,000$ Kip-ft. and $M_{xy} = 300,000$ kip-ft.

5.3 VERIFICATION OF ANALYTICAL MODELS

Verifications of analytical models were conducted by comparing computed vibrational frequencies with ambient vibration measurements by Abdel Ghaffar and Scanlan [8].

5.3.1 Global Model

The global model of the bridge is shown in Figure 5.1. The total number of active degrees of freedom is about 4000.

The first 100 modes of the complete bridge were extracted by the subspace iteration method. The first mode is the transverse vibration of the main span deck with a natural period of 20.56 sec. Most of the lower modes involving vibration of the cables and torsional response of the deck are not relevant to the seismic performance of the structure. The seismic response of the structure is mostly affected by the coupled longitudinal and vertical vibration of the tower and the side span stiffening truss. Six of these modes were identified based on

fixed-base response spectrum analysis results. There is also a close correlation of these modes with measured values.

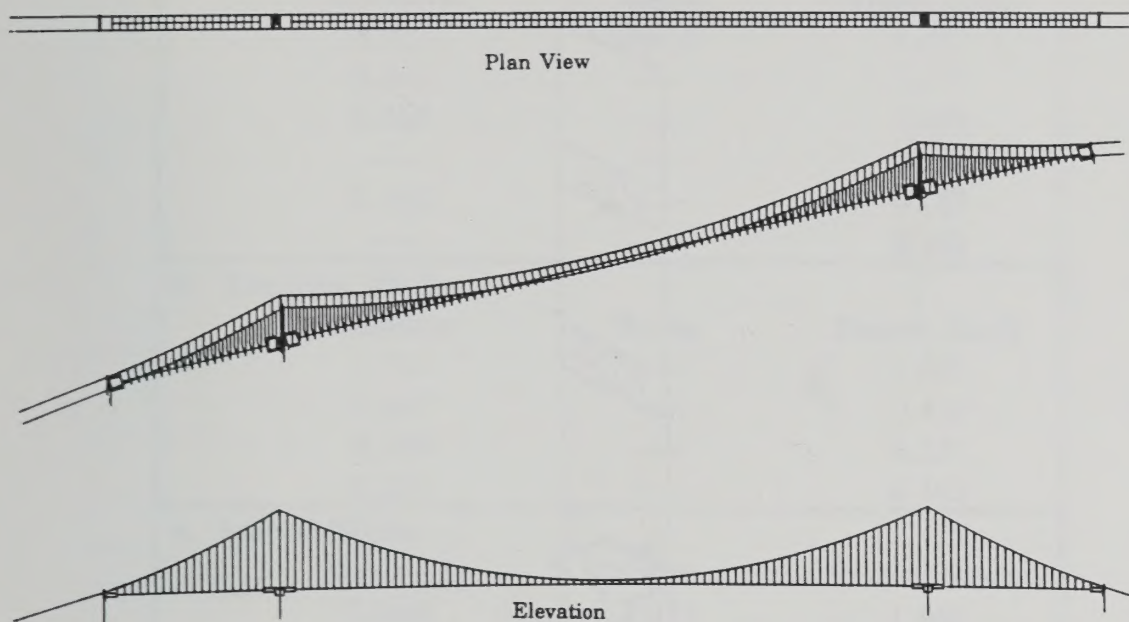


Fig. 5.1: Global Model of the Golden Gate Bridge

5.3.2 Main Tower Structure

A local main tower model is shown in Figure 5.2.

Vibration mode shapes and periods of the tower structure were computed with the tower fixed at the base and restrained at the top by equivalent cable stiffnesses. The suspended structures were not included in the model.

In the ambient vibration measurements conducted by Abdel-Ghaffar and Scanlan [8,9], the interaction between the suspended structure and the tower were identified and the authors were able to isolate those modes involving primarily vibration of the tower. These measurements provided an excellent opportunity to check the validity of the computer model for the tower structure. The vibration periods for longitudinal, torsional and transverse modes are listed in Table 5.2. Very close correlation between computed and measured values were obtained that essentially validated the computer model developed.

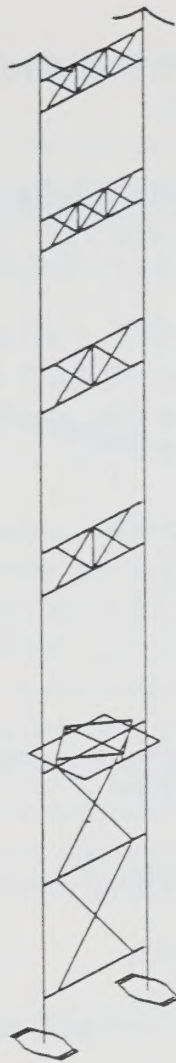


Fig. 5.2: Local Main Tower Model

5.3.3 Modal Damping Ratios

Additional information obtained from ambient vibration measurements were modal damping ratios associated with the tower modes. As shown in Table 5.3, modal damping ratios are 1.3% for the first longitudinal tower mode ($T = 1.332$ sec.); 2.2% for the first transverse tower mode ($T = 2.1845$ sec.); and 1.3% for the first torsional tower mode ($T = 1.2136$ sec.) Even higher damping ratios, up to 5%, were obtained for longer period modes ($T = 5$ sec.) involving both the tower and the suspended structure.

These values were obtained from low-amplitude ambient vibration measurements during relatively calm wind condition. Therefore, a modal damping ratio of 3% in the period range of 1 to 2 seconds can be expected.

Table 5.2: Vibration Periods of Tower

a. Longitudinal Modes			
	Measured	Baron	Present Study
	1.332	1.410	1.452
	0.546	-----	0.471
	0.266	-----	0.268
	-----	-----	0.226
	0.163	-----	0.183
	-----	-----	0.143
b. Torsional Modes			
	Measured	Baron	Present Study
	1.214	-----	1.051
	0.495	-----	0.410
	0.329	-----	0.237
	0.220	-----	0.163
c. Lateral Modes			
	Measured	Baron	Present Study
	2.1845	1.871	1.868
	0.6206	0.609	0.608
	0.3749	0.335	0.318
	0.2200	0.249	0.241
	-----	0.107	0.180

Table 5.3: Measured Modal Damping for Tower

Mode	Period (sec)	Damping Ratio
Longitudinal		
1	1.332	1.3%
2	0.546	0.6%
Transverse		
1	2.184	2.2%
2	0.621	0.8%
Torsional		
1	1.214	1.3%
2	0.495	0.5%

5.4 GLOBAL RESPONSE ANALYSIS

During an earthquake, the structure is subjected to a combination of wave types (P, SH, SV and surface waves). Typically, the effect of earthquake on the structure is expressed as a ground motion spectrum. For long structures, spatial variability of the ground motion along the length of the bridge may become important. Phase differences in support motions may affect the structural response considerably in some places.

5.4.1 Relative Support Movements

In the global analysis of the 3D model from anchorage to anchorage, both quasi-static and vibrational responses (displacements and internal forces) were calculated by the direct time integration method. The sensitivity of the structural response to the vibrational component of the total structural response can be assessed by a conventional response spectrum analysis.

The sensitivity of the structural response to relative support movements can be assessed by quasi-static analysis of the deformed configuration for unit longitudinal and vertical support displacements respectively.

Because of the flexibility of the tower in the longitudinal direction (as compared to the cable restraint at the tower top) and its sliding connection with the main span stiffening truss, the longitudinal movement at the base of one tower support affects only the side span stiffening truss and cables, and is essentially uncoupled longitudinally with the other main tower. The most significant effect of the quasi-static analysis comes from the longitudinal movement of a cable anchorage. The adjacent side span, main span and both main towers are affected.

Member internal forces caused by relative support movements of 1 ft are small due to the flexible nature of the structure. For relative support displacements contained in the multiple-support input, the quasi-static component is not important for this flexible structure.

Since there exists some degree of uncertainty in the synthetic ground displacements generated for this study, it is difficult to establish an upper bound for the relative support movement under the maximum credible earthquake. To determine the performance of the tower structure to displacement, a nonlinear analysis was conducted to establish how much relative support movement the tower structure can tolerate.

5.4.2 Rigid-Support Response Spectrum Analysis

The response spectrum method was used to obtain some estimates of the structural vibrational response. The input used was the 5% damped acceleration spectral curve corresponding to the maximum credible event. All six supports were assumed to be rigidly connected. The Complete Quadratic Combination (CQC) modal combination method was used to estimate various maximum response values.

A total of 100 modes were included in the analysis. The natural period of the highest mode was 1.2246 sec which may be too long for some critical tower members. Results for these analyses are summarized in Tables 5.4 to 5.9. The results are reported separately for each direction of earthquake input, i.e. no directional combination was performed. The objective here was to obtain some insight into the earthquake response of the bridge.

Maximum displacement responses of the suspended structure are presented in Table 5.4 and the maximum displacements of the tower at top, at Strut 3 and Strut 4 at roadway level are presented in Table 5.5.

Member forces developed in the tower shaft are presented in Table 5.6. The maximum base moment caused by the longitudinal earthquake is 2,400,000 kip-ft which is significantly higher than the base moment capacity for initial rivet shear failure of 900,000 kip-ft established earlier and indicates that the rivets would have failed in shear and partial uplifting would have occurred. Because of this softening effect, stresses at the base may be reduced. A better estimate can only be obtained from a nonlinear analysis.

The large base moment was contributed mostly by the longitudinal inertia force of the suspended side span as evidenced by the drastic increase of longitudinal shear in the tower shaft from 5,113 kip at about roadway level to 12,517 kip at the base as shown in Table 5.6. Also contributing are the high longitudinal shear in the side span to main tower connection as shown in Table 5.7.

More than 90% of the longitudinal response was caused by a single mode, the 62nd mode, with a period of 1.86 sec. This is primarily a longitudinal mode of both the tower and side span coupled together with minor vertical vibration of the side span. At top of the tower shaft, a maximum shear of 4,730 kip was induced just below the cable saddle by the longitudinal earthquake.

Table 5.4: Response Spectrum Analysis: Deck Displacements

Displacements in feet	DIRECTION OF EARTHQUAKE INPUT		
	Longitudinal	Vertical	Transverse
<i>Side Span - ½ pt.</i>			
U _L	3.16	0.43	----
U _V	0.93	5.18	----
U _T	----	----	3.75
<i>Main Span - ¼ pt.</i>			
U _L	2.60	0.04	----
U _V	2.68	3.12	----
U _T	----	----	3.31
<i>Main Span - ½ pt.</i>			
U _L	2.57	0.04	----
U _V	0.14	4.43	----
U _T	----	----	4.86

Table 5.5: Response Spectrum Analysis: Tower Displacements

Displacements in feet	Longitudinal Displacement due to Longitudinal EQ	Transverse Displacement due to Transverse EQ
<i>Top</i>	0.65	2.10
<i>Strut 3</i>	2.20	1.15
<i>Strut 5 (roadway)</i>	0.90	0.22

Table 5.6: Response Spectrum Analysis: South Tower Forces

Units kips & feet	DIRECTION OF EARTHQUAKE INPUT			Dead Load
	Longitudinal	Vertical	Transverse	
<i>Top</i>				
P	4,134	9,811	568	
V _L	4,730	565	75	
V _T	144	17	1,059	
<i>Below Strut 3</i>				
P	3,795	9,834	15,339	
V _L /M _T	1,541 / 837,090	258 / 95,327	133 / 11,075	
V _T /M _L	96 / 17,848	11 / 2,051	3,775 / 182,234	
$\sigma_{\max}$	28.7 ksi	5.1 ksi	13.2 ksi	-10.6 ksi
<i>Below Strut 4</i>				
P	4,126	9,844	29,111	
V _L /M _T	5,113 / 625,030	759 / 59,164	112 / 20,293	
V _T /M _L	215 / 7,149	37 / 1,533	4,956 / 376,289	
$\sigma_{\max}$	14.5 ksi	2.9 ksi	21.8 ksi	-9.3 ksi
<i>Base</i>				
P	4,009	10,090	56,745	
V _L /M _T	12,517 / 2,406,563	928 / 211,957	354 / 52,417	
V _T /M _L	209 / 6,426	353 / 7,081	5,835 / 252,304	
$\sigma_{\max}$	26.1 ksi	3.2 ksi	9.3 ksi	-6.6 ksi

Table 5.7: Response Spectrum Analysis: Wind Lock Forces

Forces in kips		DIRECTION OF EARTHQUAKE INPUT		
		Longitudinal	Vertical	Transverse
<i>South Side Span to Tower</i>				
	V _L	16,030	1,488	40
	V _T	----	----	1,402
<i>Main Span to South Tower to North Tower</i>				
	V _T	----	----	986
	V _T	----	----	1,018
<i>North Side Span to Tower</i>				
	V _L	15,839	1,311	40
	V _T	----	----	1,482

Maximum additional cable forces induced by the vertical earthquake excitation were 9,897 kip at an anchorage and 10,539 kip near the main tower as shown in Table 5.8. The maximum cable force induced by the longitudinal earthquake at an anchorage was 8,184 kip.

Table 5.8: Response Spectrum Analysis: Cable Forces

Forces in kips	DIRECTION OF EARTHQUAKE INPUT		
	Longitudinal	Vertical	Transverse
Main Cable (A = 832 sq. in.)			
<i>South Anchorage</i>	7,572	9,897	567
<i>South Tower / South</i>	7,116	10,539	596
<i>South Tower / North</i>	1,841	9,767	550
<i>North Tower / South</i>	2,803	9,706	525
<i>North Tower / North</i>	8,069	10,423	555
<i>North Anchorage</i>	8,184	9,739	513
Cable Tie-Downs			
<i>South</i>	393	369	40
<i>North</i>	384	412	40

Table 5.9: Response Spectrum Analysis: Stiffening Truss Forces

Units kips & feet	DIRECTION OF EARTHQUAKE INPUT		
	Longitudinal	Vertical	Transverse
<i>½ Side Span</i> P	8,460	809	-----
V ₂ /M ₃	267 / 73,402	48 / 106,868	-----
V ₃ /M ₂	-----	-----	193 / 512,867
σ _{max}	37.7 ksi	27.0 ksi	30.0 ksi
<i>¼ Main Span</i> P	540	77	-----
V ₂ /M ₃	59 / 23,609	400 / 83,277	-----
V ₃ /M ₂	-----	-----	618 / 225,580
σ _{max}	4.6 ksi	13.3 ksi	8.5 ksi
<i>½ Main Span</i> P	56	115	-----
V ₂ /M ₃	105 / 6,386	65 / 113,691	-----
V ₃ /M ₂	-----	-----	141 / 273,805
σ _{max}	1.3 ksi	20.8 ksi	12.4 ksi

Maximum forces and stresses in the suspended structure are listed in Table 5.9. The significant dynamic stresses induced in the side span truss chord deserve careful attention.

Results of the response spectrum analysis provided some insights on the structural behavior and revealed some potential vulnerabilities. However, because the number of modes included was not sufficient, particularly for the transverse earthquake excitation, results could be unconservative in some cases. These results nevertheless provided a good check on the time history analysis.

5.4.3 Global Response Analysis to Multiple-Support Inputs

Step-by-step integration of the coupled equation of motion was used to solve the multiple-support excitation analysis. The structure was subjected simultaneously to a three component input at each of its six supports; both acceleration and displacement time histories were specified. The total duration of the analysis was 60 seconds with an integration time step of 0.04 second.

a. Structural Damping

The Raleigh damping matrix in the form of

$$C = \alpha M + \beta K$$

was used in the time history analysis. M and K are mass and stiffness matrices. The values of coefficients α and β were determined to yield a damping ratio of 5% at two periods, $\frac{1}{2}$ sec and 20 sec, corresponding to the second longitudinal mode of the isolated tower and the first transverse mode of the main span suspended structure. At the period of 1 second, the damping ratio was 2.7% and decreased to 1.7% for a period of 2 seconds. These damping values are consistent with ambient vibration measurements reported earlier.

b. Displacement Response

Suspended Structure

Maximum vibration displacements of the suspended structure are presented in Table 5.10. At center of the main span, the deck experienced lateral displacement of 11.76 ft and vertical displacement of 6.76 ft which are not unusual. The maximum longitudinal displacement of the main span was 2.3 ft, which is about 53% higher than the one-way movement of 18 in. provided by the expansion joints. The maximum longitudinal displacement of the side spans was 3.2 ft on the San Francisco side and 3.75 ft on the Marin side. Both movements exceed the capacity of the side span expansion joints.

Table 5.10: Time History Analysis: Maximum Deck Displacements

Displ: feet Time: seconds	Longitudinal		Vertical		Transverse	
	Displ.	Time	Displ.	Time	Displ.	Time
<i>South Side Span</i> <i>½ pt.</i>	3.203	11.88	4.176	55.60	3.930	10.84
<i>Main Span</i> <i>¼ pt.</i>	2.295	32.56	5.770	50.28	7.585	27.00
<i>½ pt.</i>	2.281	32.60	6.763	50.56	11.760	36.44
<i>¾ pt.</i>	2.313	32.60	4.757	40.88	7.786	27.08
<i>North Side Span</i> <i>½ pt.</i>	3.742	12.28	4.345	49.52	3.468	10.92

Main Towers

The vibrational and total longitudinal displacement time histories of the south tower and the north tower are shown in Figure 5.3 and 5.4. For each tower, displacements at three elevations are shown: top, Strut 3, and roadway level. After about 30 seconds, the vibrational components gradually vanish, and the tower moves essentially with the ground as a rigid body. Maximum displacements along the elevation of both towers are shown in Figure 5.5 for longitudinal and Figure 5.6 for transverse components.

The maximum longitudinal displacement occurred at Strut 3. The response at the north tower was significantly higher than the south tower, a result that was not expected from the response spectrum results. It is possible that due to the frequency content difference in the multiple time history input, non-symmetric modes are excited on the Marin side. The vertical input at the North anchorage contains significantly higher energy in the period range below 1.2 sec than the vertical input at the south anchorage. Since longitudinal and vertical responses are coupled, this difference in vertical input may have contributed to the difference in longitudinal tower response. Since it is equally likely that there might be stronger input on the San Francisco side, both towers should be evaluated based on the maximum response at the north tower.

Maximum transverse displacements in both towers were very similar.

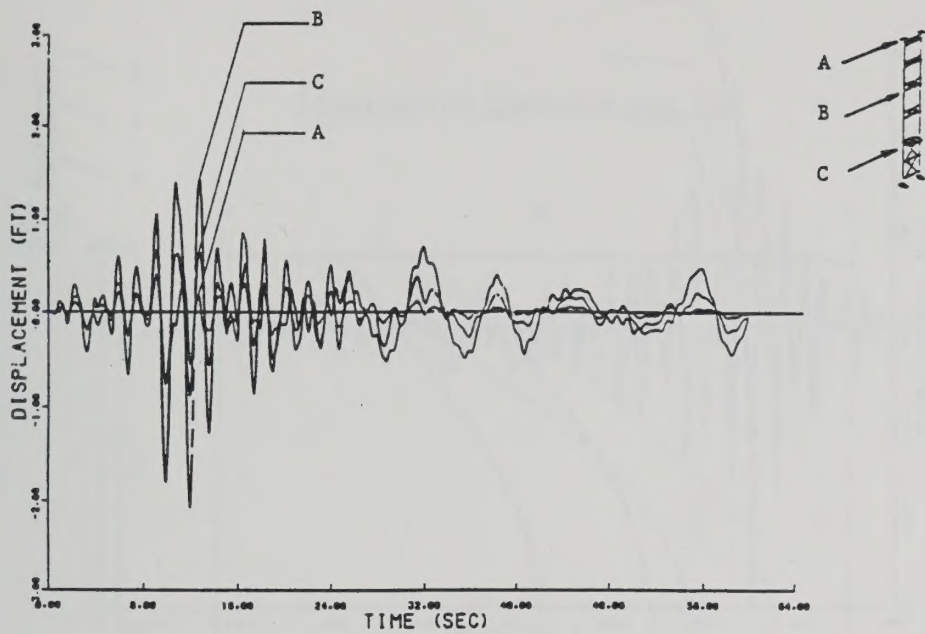


Fig. 5.3a: South Tower Longitudinal Vibrational Time History

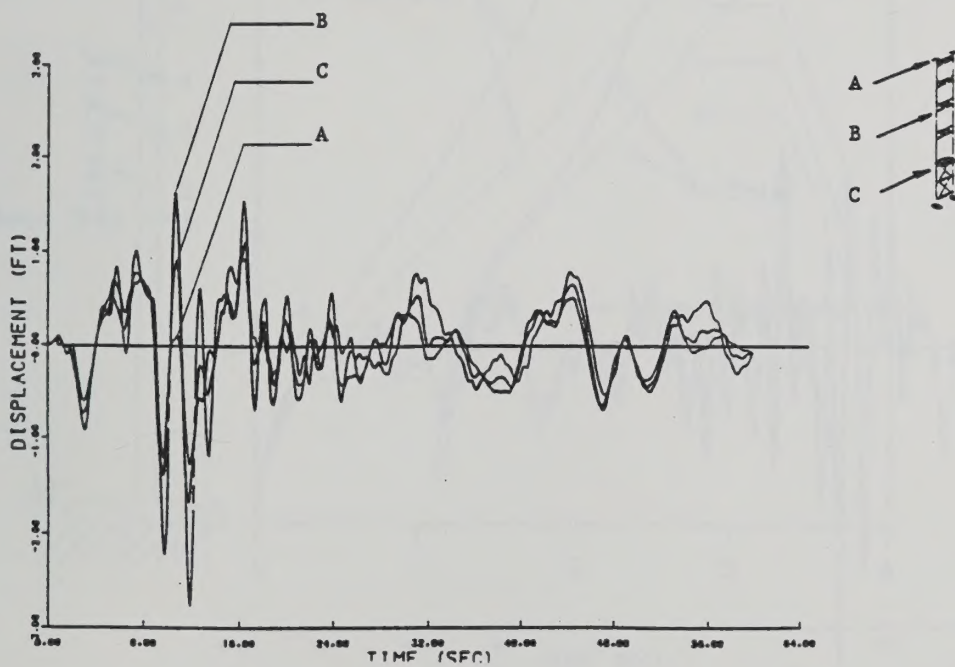


Fig. 5.3b: South Tower Total Response Time History

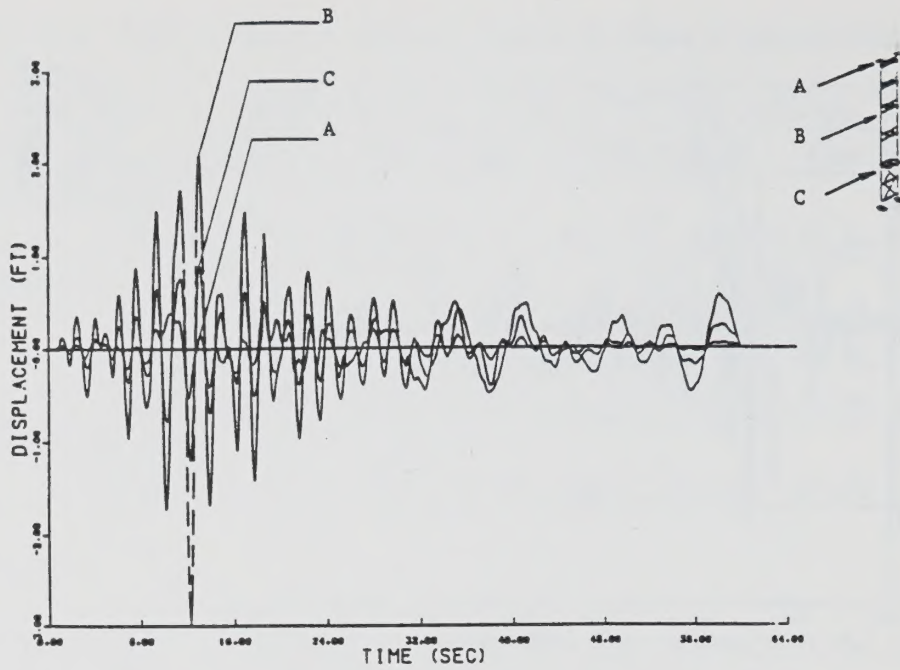


Fig. 5.4a: North Tower Longitudinal Vibrational Time History

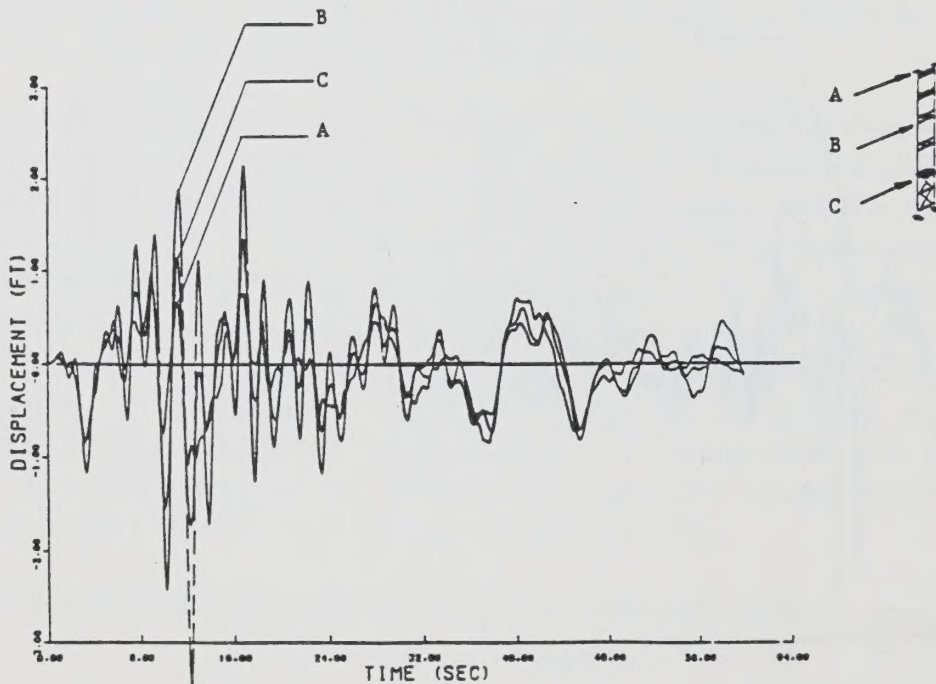


Fig. 5.4b: North Tower Total Response Time History

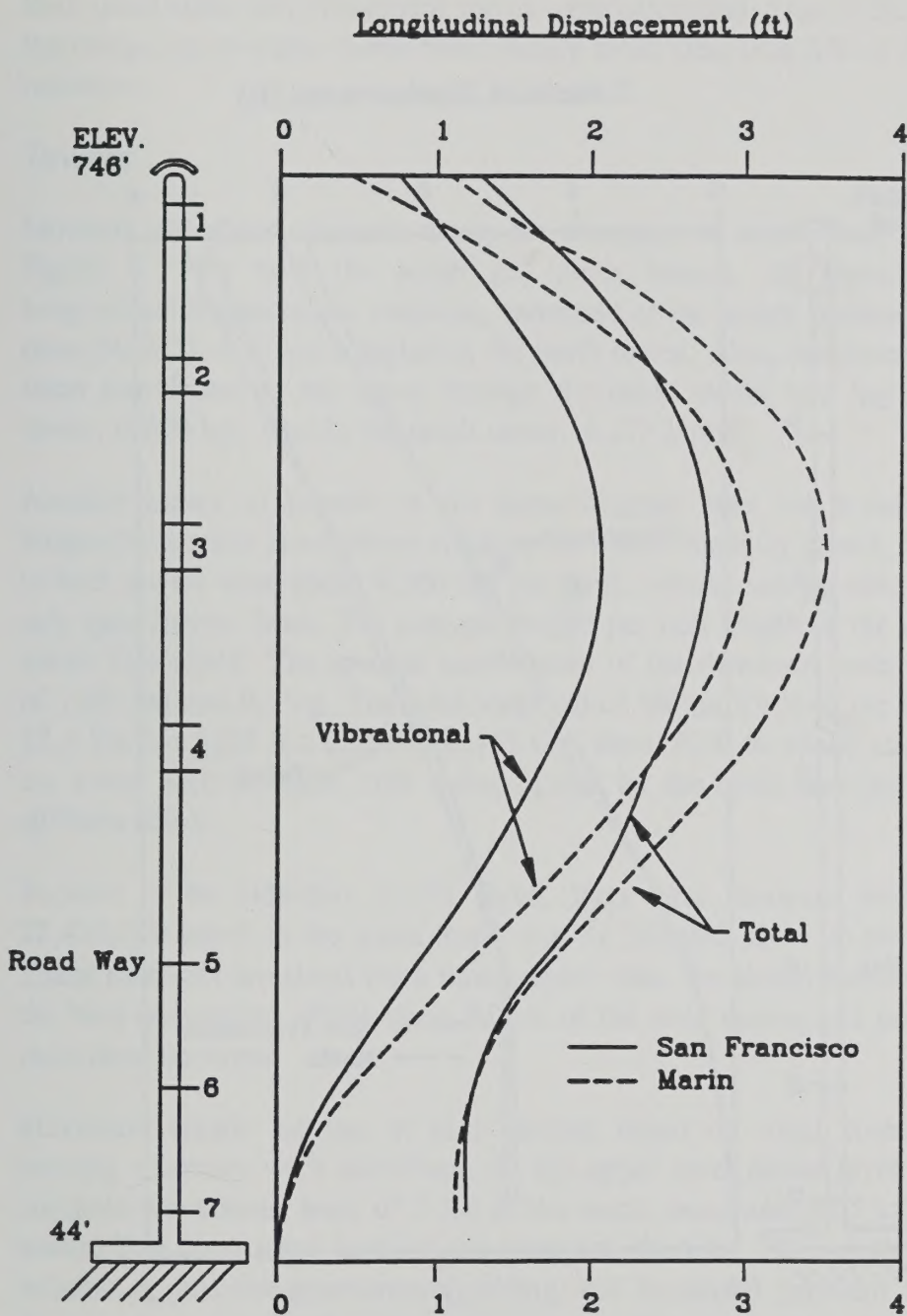


Fig. 5.5: Longitudinal Tower Displacements

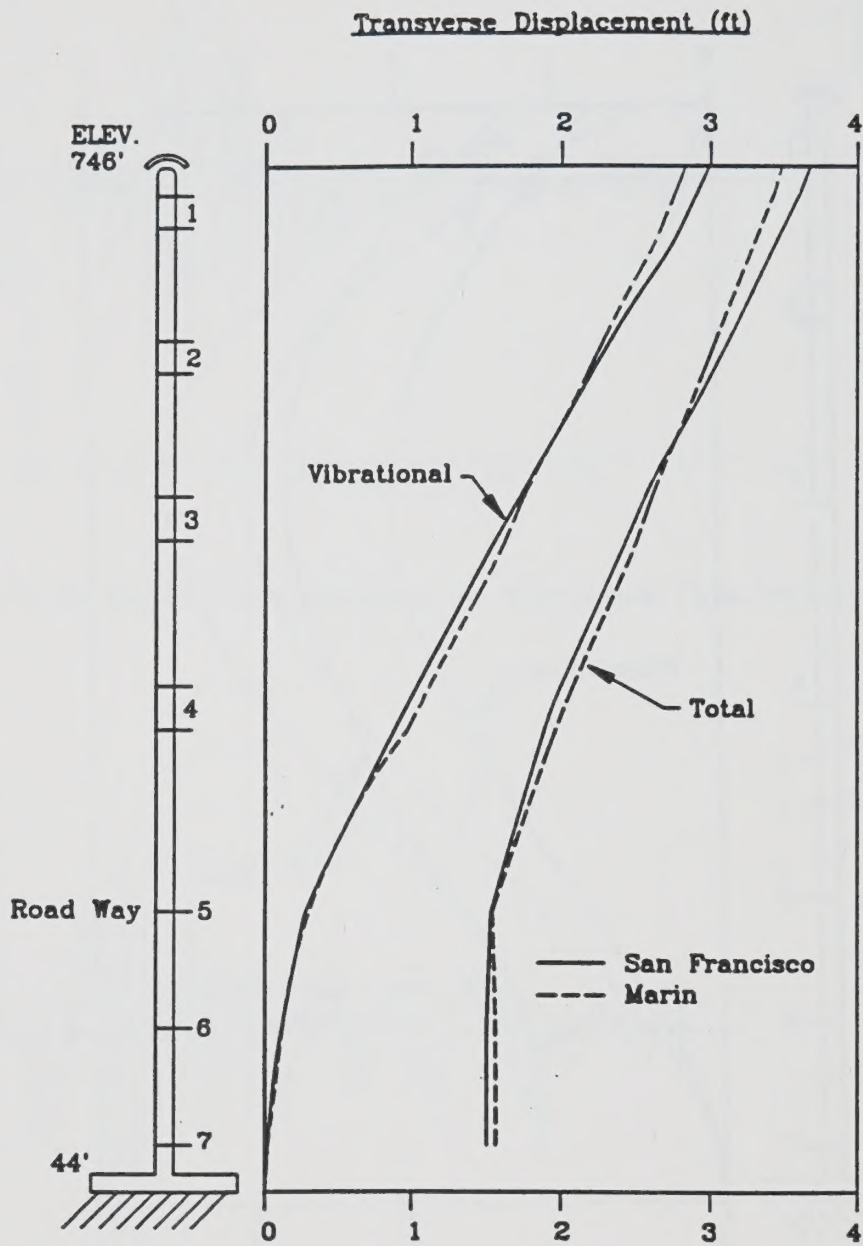


Fig. 5.6: Transverse Tower Displacements

c. Member Forces

Both quasi-static and vibrational forces were calculated. Due to the flexibility of the bridge, quasi-static forces were mostly small (less than 5% of the total force response).

Towers

Moment and shear diagrams along the elevation of tower shaft are shown in Figure 5.7 for both the south and north towers. As expected from the longitudinal displacement response, moments at the upper portion of the tower (near Strut 3) were much higher in the north tower. Also, maximum longitudinal shear transferred to the tower through the cable saddle was high in the north tower, 6,690 kip, than in the south tower, 4,123 kip).

Another aspect of interest in the shear diagram was the drastic increase in longitudinal shear from above roadway to below roadway levels. The increases in both towers were about 8,300 kip per shaft, which must be contributed by the side span inertia force. The average weight per unit length of the side span was about 12.4 kip/ft. The spectral acceleration of the dominant node with a period of 1.86 sec was 0.75 g. The total longitudinal inertia force of the side span was $12.4 \text{ kip/ft} \times 1125 \text{ ft} \times 0.75 = 10,461 \text{ kip}$, about 80% of which was delivered to the tower with the other 20% being carried by the cable through its geometric stiffness effect.

Because of the side-span inertia force, large base moments were developed: 22,450,000 kip-ft in the south tower and 31,500,000 kip-ft in the north tower. These moments are about three times greater than the elastic moment capacity of the base connection. Rivet shear failure of the hold downs and partial uplifting must have occurred.

Maximum elastic stresses at each section based on axial force and biaxial bending moments were calculated. At the upper level above Strut 3, maximum compressive stresses were 67.5 ksi in the north tower and 46.5 ksi in the south tower. The yield stress level of the plate was 50.5 ksi. The north tower results indicate a potential overstress (yielding and buckling) problem from Strut 1 down to Strut 4 under the given maximum credible earthquake scenario.

Further down the shaft, at the level just above Strut 7, very high stresses were obtained: 79 ksi at the north tower and 63 ksi at the south tower. These stresses were partly due to the reduction in shaft cross section.

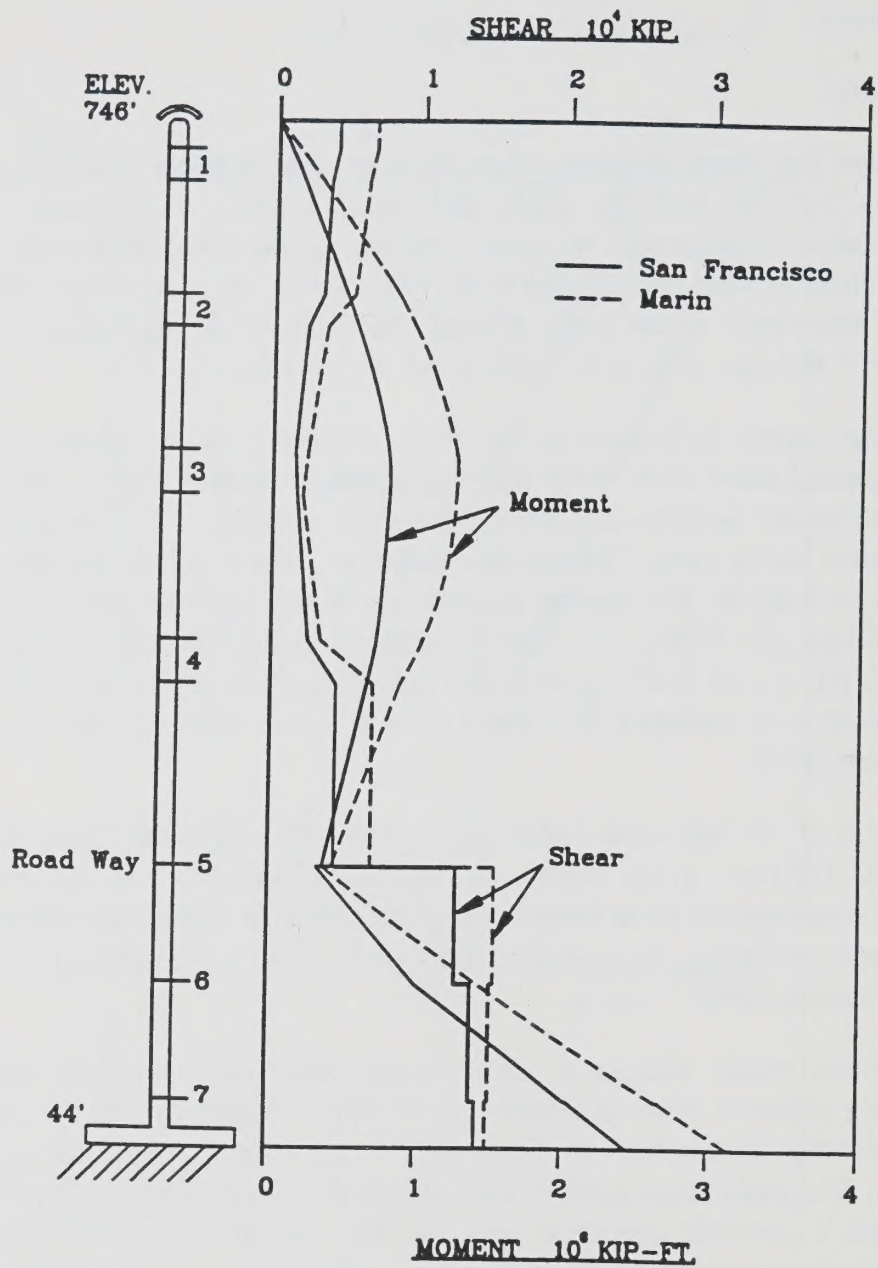


Fig. 5.7: Tower Moments and Shears

Hinge Connections

Maximum forces developed in the connections of stiffening trusses to towers and pylons are shown in Table 5.11. Transverse shear forces ranged from 1,679 kip at the north pylon to a maximum 2,467 kip (from the main span to the north tower). The longitudinal force from side spans to main towers were 18,700 kip on the Marin side and 15,380 kip on the San Francisco side.

Table 5.11: Time History Analysis: Maximum Wind Lock Forces

Forces in kips	Longitudinal	Transverse
<i>Pylon - South Side Span</i>	0	1,958
<i>South Side Span - Tower</i>	15,380	2,354
<i>South Tower - Main Span</i>	0	2,109
<i>Main Span - North Tower</i>	0	2,476
<i>Tower - North Side Span</i>	18,700	2,158
<i>North Side Span - Pylon</i>	0	1,679

Stiffening Trusses

Maximum forces in the stiffening truss superelements and the corresponding truss chord stresses are shown in Table 5.12. In the middle of the side spans, the maximum compressive stress was 55.93 ksi on the San Francisco side and the maximum tensile stress was 53.5 ksi on the Marin side. The stresses in the main span were all well below yield stress.

Table 5.12: Time History Analysis: Stiffening Truss Forces

V: vertical axis T: transv. axis	Time (sec)	Axial (kips)	Moment-V (kip-ft)	Moment-T (kip-ft)	Stress (ksi)	
South Span ½ pt.	11.8	-7,947	120,400	-38,360	38.77	T
	10.92	6,596	-608,600	22,330	-55.93	C
Main Span ¼ pt.	50.52	-210	-88,930	-98,330	16.68	T
	9.8	428	204,700	-3,221	9.06	C
½ pt.	50.64	-71.4	82,720	123,400	22.53	T
	17.76	53.5	-375,800	1,672	-17.34	C
¾ pt.	9.64	-455.5	73,040	-4,737	4.19	T
	52.52	181	91,820	103,900	-17.45	C
North Span ½ pt.	11.32	-10,640	-343,300	-39,200	53.50	T
	21.32	3,351	303,500	94,700	-44.38	C

Pylons

Maximum longitudinal and transverse base shear at pylons are shown in Table 5.13. Nominal shear stresses were calculated. The demand on shear was much higher on the San Francisco side. The nominal shear capacity was calculated based on concrete contribution alone using:

$$\nu_c = 2 \left[1 + \frac{N_u}{2000A_g} \right] \sqrt{f'_c} \quad \text{for axial compression } N_u > 0$$

$$\nu_c = 2 \left[1 + \frac{N_u}{500A_g} \right] \sqrt{f'_c} \quad \text{for axial tension } N_u < 0$$

Table 5.13: Time History Analysis: Pylon Base Shear

	South Pylon	North Pylon
<i>Demand (kips)</i>		
V_L	21,730	10,990
V_T	15,920	12,160
V_{Total}	26,938	16,390
P_{axial}	5,906 T	5,390 C
<i>Capacity (kips)</i>		
$f'_c = 3000$	5,394	12,126
$f'_c = 6000$	7,629	17,149
<i>D/C Ratio</i>		
$f'_c = 3000$	5.00	1.35
$f'_c = 6000$	3.57	0.95

As shown in the Table, the Demand/Capacity ratio was 3.6, if the concrete strength f'_c of 6,000 psi is used. This result indicates the possibility of concrete shear failure when ground acceleration reached $0.3 \times 0.65g = 0.20g$, a level slightly lower than the lower level earthquake (average return period = 72 years), and indicated the urgent need to strengthen these structures. (During the Loma Prieta earthquake, peak ground acceleration of $0.1g$ was recorded in the San Francisco area.) Any failure of the pylons may particularly endanger the cable tie-down and therefore the main suspension span.

Cable, Suspenders and Cable Tie-downs

Maximum additional cable forces due to earthquake effect are shown in Table 5.14 with a maximum of 10,480 kips occurring in the San Francisco side span. These forces correspond to a stress level of 12.6 ksi and about 20% of the initial

horizontal cable tension ($H_o = 53,500$ kip), which is high enough to warrant attention.

Table 5.14: Time History Analysis: Cable Forces

Forces in kips	Vibrational		Total	
Main Cable (A = 832 sq. in.)				
<i>South Anchorage</i>	8,849 kips	10.6 ksi	9,194 kips	11.0 ksi
<i>South Tower / South</i>	10,130 kips	12.2 ksi	10,480 kips	12.6 ksi
<i>South Tower / North</i>	7,847 kips	9.4 ksi	8,148 kips	9.8 ksi
<i>North Tower / South</i>	7,289 kips	8.8 ksi	7,460 kips	9.0 ksi
<i>North Tower / North</i>	7,068 kips	8.5 ksi	7,111 kips	8.5 ksi
<i>North Anchorage</i>	7,801 kips	9.4 ksi	7,888 kips	9.5 ksi
Cable Tie-Downs				
<i>South</i>			951 kips	
<i>North</i>			578 kips	

Maximum force in the cable tie-downs was 951 kip. The maximum suspender forces are shown in Table 5.15.

Table 5.15: Time History Analysis: Cable Forces

	Axial (kips)	Time (sec)
<i>South Side Span</i> ½ pt.	107	18.40
<i>Main Span</i> ¼ pt.	62	49.88
½ pt.	89	50.80
¾ pt.	61	52.16
<i>North Side Span</i> ½ pt.	85	49.64

5.5 INTERPRETATION

In the preceding section, the potentially vulnerable details in a seismic event were identified using linearized global response analyses performed for the maximum credible earthquake. It is equally important to assess the structural response of the bridge under more frequent earthquakes such as the upper level and lower level events. Over the period range of interest, from 0.6 seconds to 2.0 seconds, the spectral acceleration of the upper level event is, on the average,

60% that of the maximum credible earthquake. The fraction drops to 25% for the lower level event.

Without further analysis, some preliminary conclusions can be drawn in regard to the structural response under these more frequent occurring earthquake excitations. For example, it was estimated that the demand/capacity ratio for concrete pylon shear was 3.6. Assuming that the fundamental period of the pylon falls in the range of 0.6 sec to 2 sec. This Demand/Capacity ratio indicates that with earthquakes at slightly higher levels than the lower level event, some damage in pylon concrete, due to shear may occur.

For strongly nonlinear behavior such as the base connection of the steel towers, structural response evaluations should be based on displacement response using nonlinear analysis procedures as discussed in the following section.

5.6 NONLINEAR RESPONSE ASSESSMENTS OF TOWER

The tower may be considered to act as a nonlinear system not only geometrically but materially. The nonlinear moment-rotation relationship for the tower base connection is shown in Figure 5.8. When the yield rotation is defined as the point of initial hold down rivet shear failure, and the ultimate rotation is defined as the point when extreme outside filaments reach buckling stress level, the rotational ductility factor of the base section is about 7. The nonlinearity of the tower base needs to be accounted for so that more realistic estimates of the stresses in the steel shaft and bearing stresses on top of the concrete pier can be established.

Another unique aspect of importance is the large base dimension of the shaft (54 ft by 33 ft) and the concrete pier (90 ft by 134 ft). The weight of the structure itself and the size of the base provide a certain amount of resistance against overturning and uplifting.

During the 1989 Loma Prieta earthquake, rocking response occurred in the Pier E17 of the Bay Bridge as described in the Report of the Governor's Board of Inquiry [1]. The base overturning moment apparently reached yield level and cause some concrete fracture and yielding of vertical steel bars. However, due primarily to the large base dimensions (17'-10" by 20'-10") and stability provided by the weight of the structure the integrity of the pier and the structure was not affected much. The Bay Bridge experience seems to suggest some rocking and uplifting of massive structure should be allowed as long as we can assure that responses at design level earthquake input remain satisfactory.

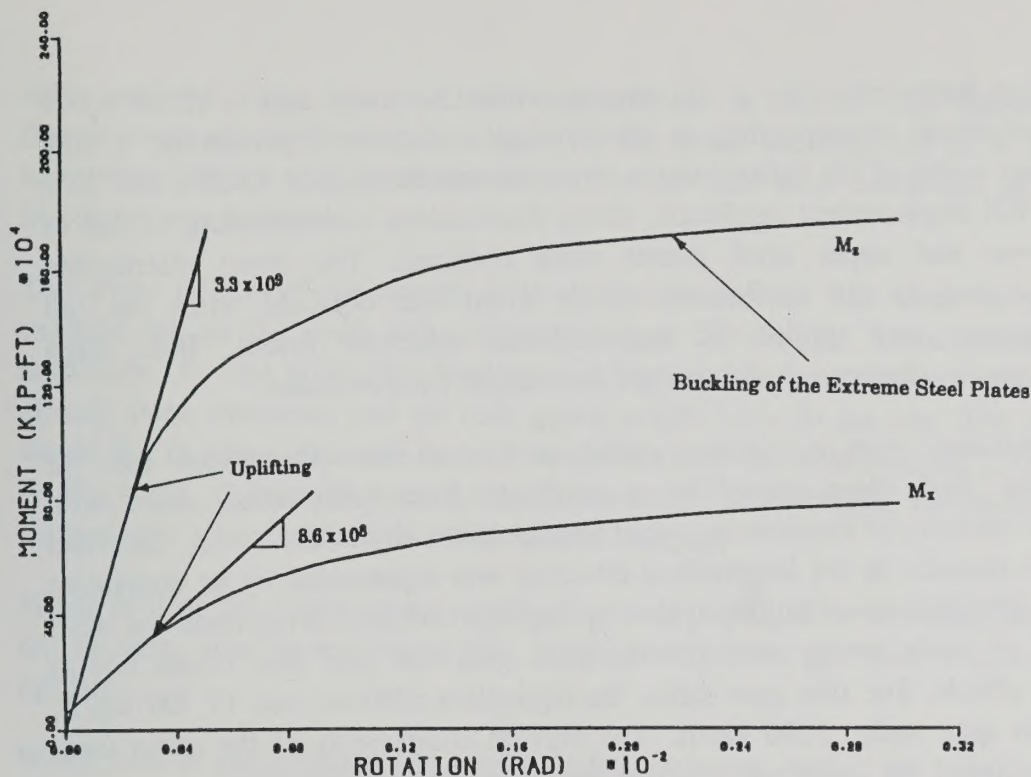


Fig. 5.8: Moment-Rotation at Base of Tower

5.6.1 Analytical Approach

Incremental nonlinear analyses of the tower structure subjected to imposed nodal displacements were conducted to establish the degree of damage at the tower base under the specified maximum credible earthquake input, to establish the threshold earthquake level that may initiate damage to structural components, and to establish the capacity of the tower for relative support movements.

It is generally held that the displacement response of a yielding nonlinear long period structure is approximately the same as that of a linear structure, provided they both have the same initial elastic stiffness. However, internal forces will be quite different in the two systems.

In the global analysis described above, correct linearized rotational stiffnesses were specified at the tower base consistent with the moment-rotation curves. Therefore, displacement responses obtained for the tower structure will be representative of that in a truly nonlinear (material and geometric) analysis. The displacement response so computed could be scaled based on ground acceleration input (g-level) or more properly, based on the spectral acceleration value averaged over the period range of primary structural participation. Displacement responses of the structure due to the upper level earthquake could be obtained by simply scaling the results by 60%; and, for displacement responses due to the lower level event, by a scale factor of 25%.

To obtain better estimates of the stresses within the tower and to obtain tower force responses corresponding to the varying earthquake input levels, a fully nonlinear model of the isolated tower structure was developed. At 25% and 60% of the full displacement amplitude, stress distributions corresponding to that of the lower and upper level events were obtained. The stress distribution corresponding to the maximum credible event was obtained when the full displacement was applied to the nonlinear structure model. Both large displacement effect and nonlinear base connection were included.

The nonlinear base connection under each shaft was discretized into 42 filaments, each characterized by a nonlinear force-deformation curve with unequal tension and compression yield levels. At the top of the tower, the effect of cable restraint in the longitudinal direction was represented by an equivalent spring. The stiffness of this equivalent spring was obtained to represent the same amount of strain energy stored in the main span and side span cables due to seismic effects. For side span cable, the equivalent stiffness was 15,700 kip/ft; for main span cable, 5200 kip/ft. The vertical components of the cable force were applied at the tower top to ensure the proper $P-\delta$ moment effect.

In considering the maximum displacement responses of the tower, maximum longitudinal and transverse displacements were considered separately.

5.6.2 Assessments of the Nonlinear Tower Response

Displacement response of the main towers was based on the global response analysis of the main bridge under the multiple-support time history input.

The longitudinal response of the north tower was much higher than that of the south tower while the transverse responses in both towers were practically the same. The longitudinal response of the south tower based on time history analysis was very close to the results obtained from the rigid-base spectrum analysis. The differences in the multiple-support inputs input may have excited certain modes more strongly in the time history analysis, resulting in a higher response on the north tower. Since uncertainty in ground motion input is high and since it is equally likely that in a different scenario, the seismic input on the San Francisco side may be higher, both towers should be evaluated based on the worst case.

a. Longitudinal Response of the Tower

In the deformed configuration of the north tower, the maximum vibrational displacement in the longitudinal direction of about 3 ft occurred at Strut 3. At the roadway level, the maximum displacement was about 1.2 ft. At the top, the

cable restraint limits the maximum displacement to 0.6 ft. The vibrational displacement amplitude along the height of the tower was imposed in 50 equal increments. forces and stress distributions at the base were calculated as each increment was imposed.

The shear failure of the riveted hold down connections first occurred at an extreme cell on the south side corresponding to 26% of the full displacement amplitude. At that point, the dead load at the two southern rows of cells had already been overcome and all hold down angles were in tension. The rivets would fail in shear when the normal tensile stress in the cells reached 2.97 ksi. At this stage, all inner four rows of cells are still in compression. The maximum compressive stress in the cells of the base is 14.6 ksi.

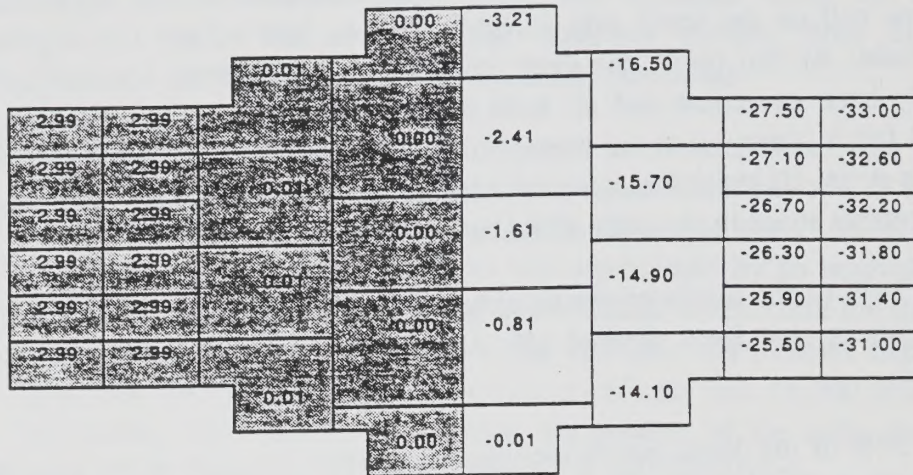
When 34% of the displacement amplitude was imposed, all rivet connections at the southern side were sheared off. About one third of the base section was uplifted.

When 58% of the displacement amplitude was imposed, half of the base section was uplifted. The maximum compressive stress in the cell was 23.8 ksi. At this stage, the base section is essentially plastic and provides little additional rotational stiffness.

The maximum stress in the cells at the base when the full displacement amplitude was imposed was 33 ksi in compression as shown in Figure 5.9 with the stresses being redistributed significantly.

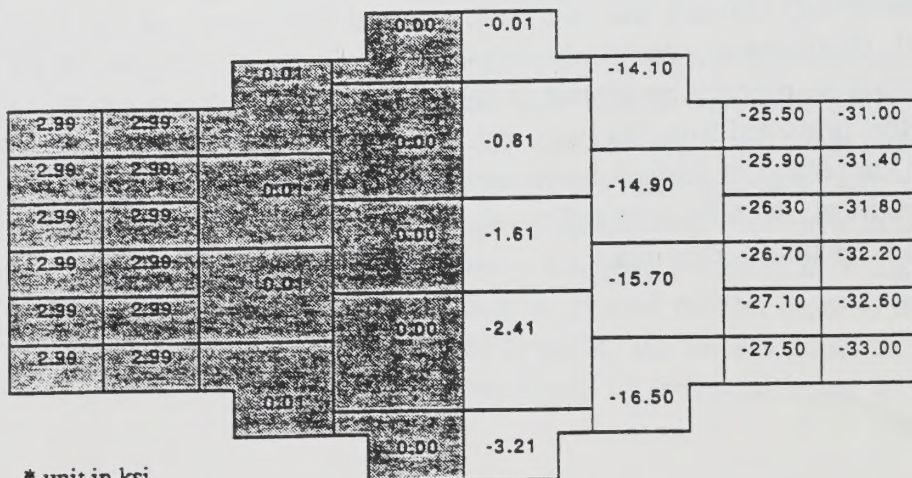
The relationship between the total base moment developed and the fraction of imposed displacement amplitude imposed is shown in Figure 5.10. The maximum base moment developed at the base in the nonlinear structure is about half of that predicted from the linear elastic analysis. Figure 5.11 shows the base rotation developed at various levels and the corresponding rotational ductility. The rotational ductility demand at the tower base was about 4 under the maximum credible earthquake.

**** West Shaft ****



 Rivet failed in shear or Base plate uplift

**** East Shaft ****



* unit in ksi

Fig. 5.9: Stress Distribution at the Tower Base Under Full Longitudinal Displacement

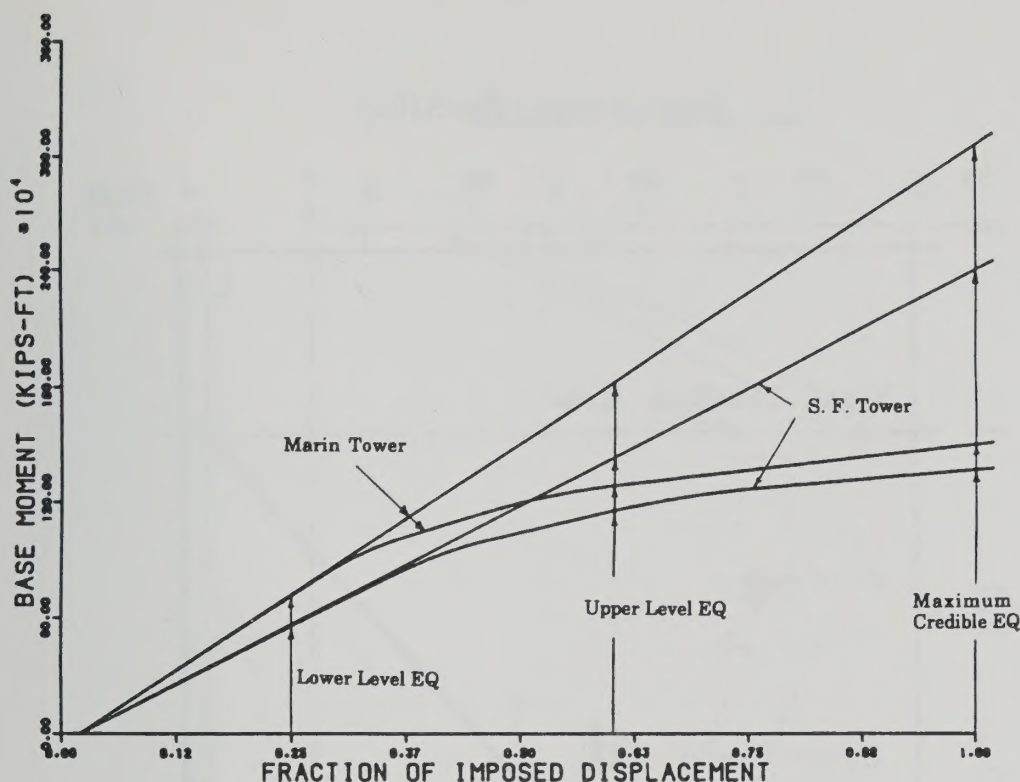


Fig. 5.10: Tower Base Moments for Different Earthquake Levels

Under the lower level event (25%), the base connection remains essentially undamaged. The first shear failure of the hold down rivet connection occurred at a level slightly above the lower level event. Under the upper level event (about 60% of displacement amplitude), almost all rivet connections would fail and significant rocking and uplifting responses would occur. However, the maximum compressive stress in the web plate was low (24 ksi), which is much lower than the elastic stress predicted, primarily because of the softening effect at the base. Similarly, it may be concluded that the concrete bearing stress under the steel base plate would be reduced to about 2.46 ksi (from a maximum of 4.73 ksi). If on the other hand, the base connection is strengthened and remains elastic, much higher stresses will be induced in both steel web plate and in the concrete pier as indicated by the elastic analysis.

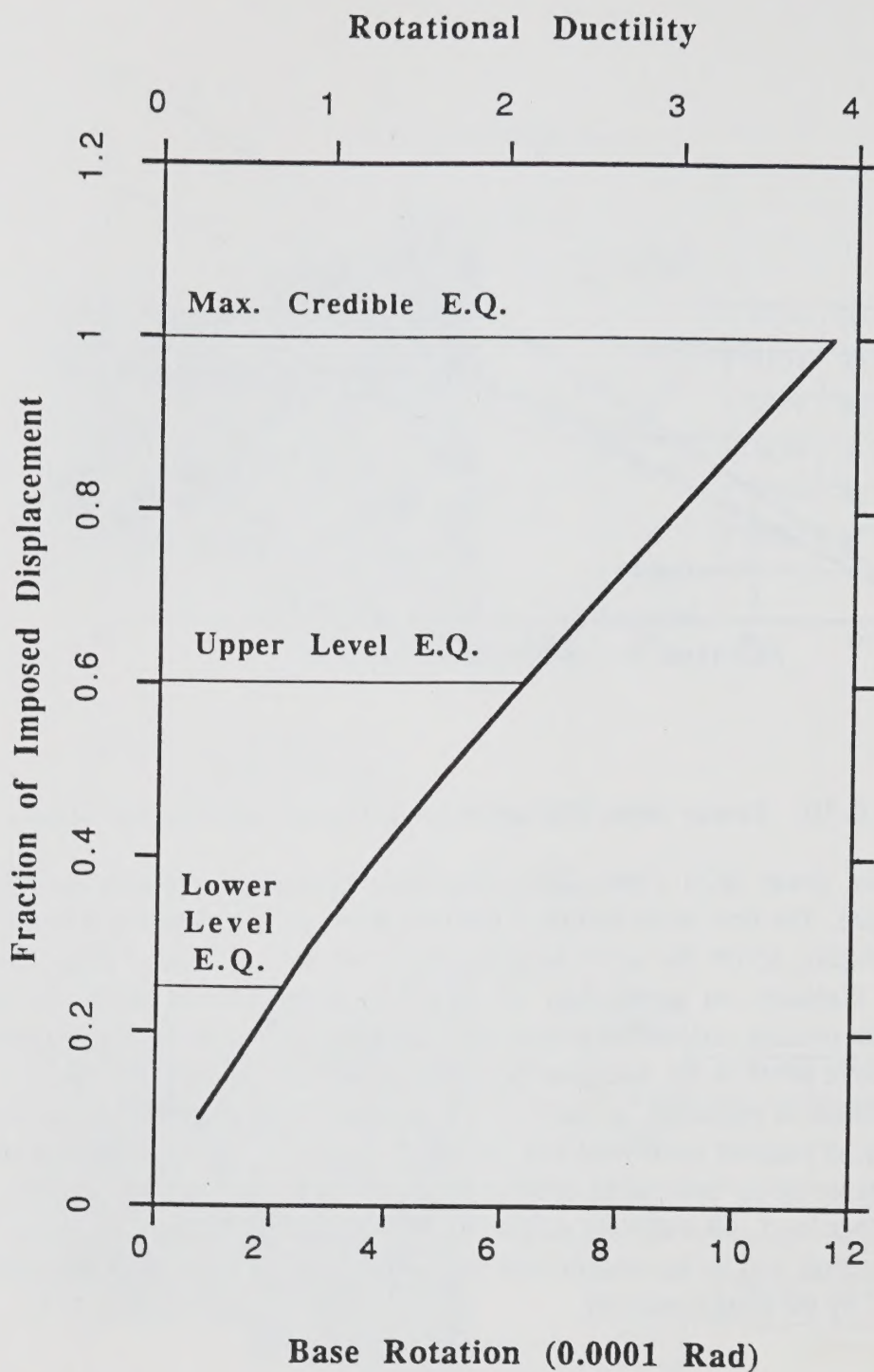


Fig. 5.11: Rotational Ductility Demands for Different Earthquake Levels

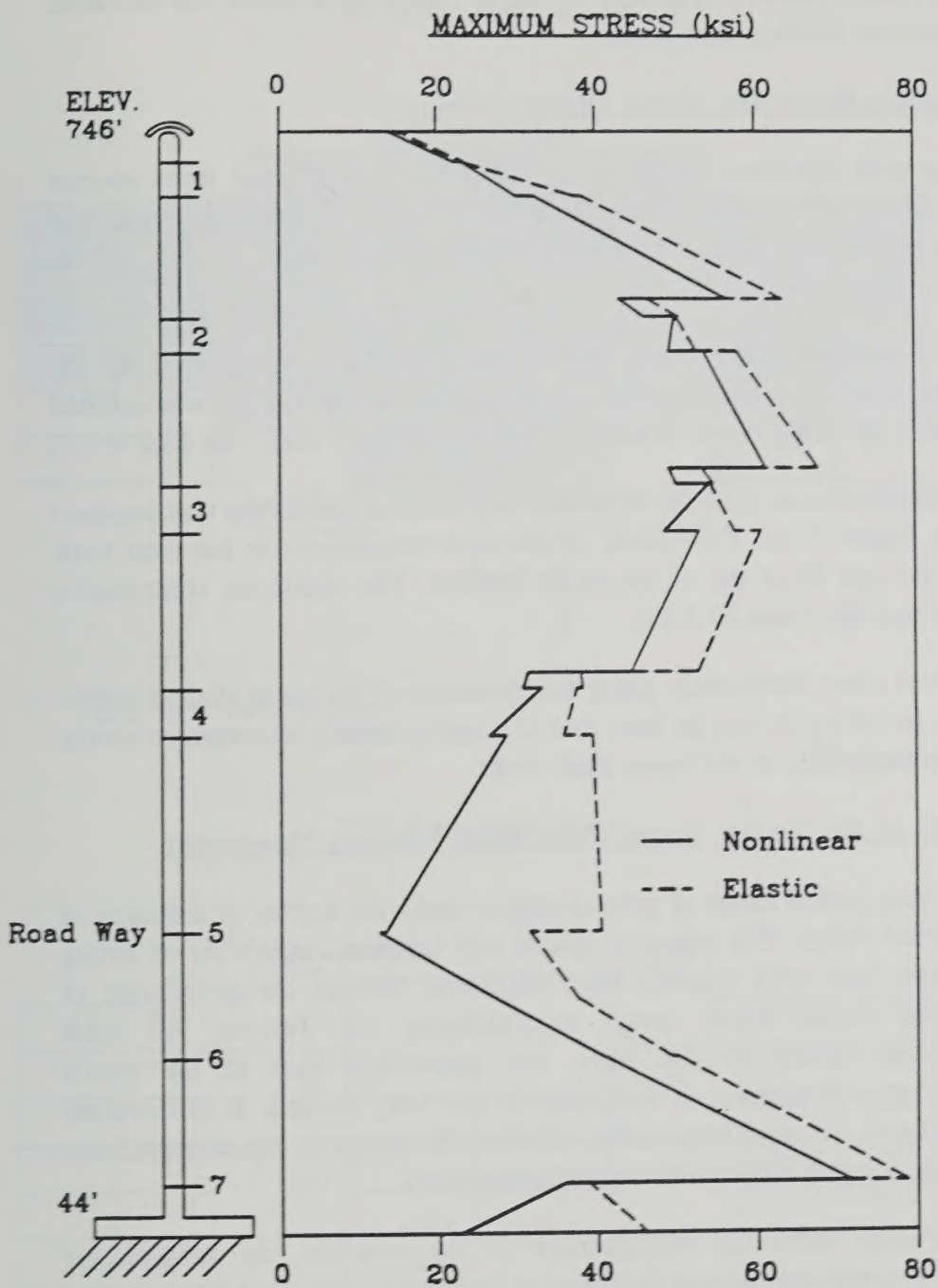


Fig. 5.12: Maximum Longitudinal Tower Stresses

The maximum stress distribution along the elevation of the tower is shown in Figure 5.12 along with the maximum elastic stress directly obtained from the time history results. The maximum stress at the base is reduced to about half of the elastic stress. As one moves up the elevation, the base softening effect becomes less important. At the roadway level the maximum stress in the shaft is

governed by the transverse response. At upper levels, both linear and nonlinear analyses produce similar stress levels.

b. Transverse Response of the Tower

In the transverse direction, the tower acts as a frame with axial force couples carrying a high portion of the total overturning moment. Uplifting of the base plate occurred initially at the west edge of the west shaft when 52% of the displacement amplitude was imposed.

First hold-down rivet connection shear failure occurred when 78% of the displacement amplitude was imposed. About half of the west shaft was uplifted and the maximum compressive stress at the base of the east shaft was 17.2 ksi.

The stress distribution at the base when full displacement amplitude was imposed is shown in Figure 5.13. Two-thirds of the rivet connections in the west shaft failed in shear and 30 of the 42 filaments uplifted. The maximum compressive stress in the east shaft was 20.3 ksi.

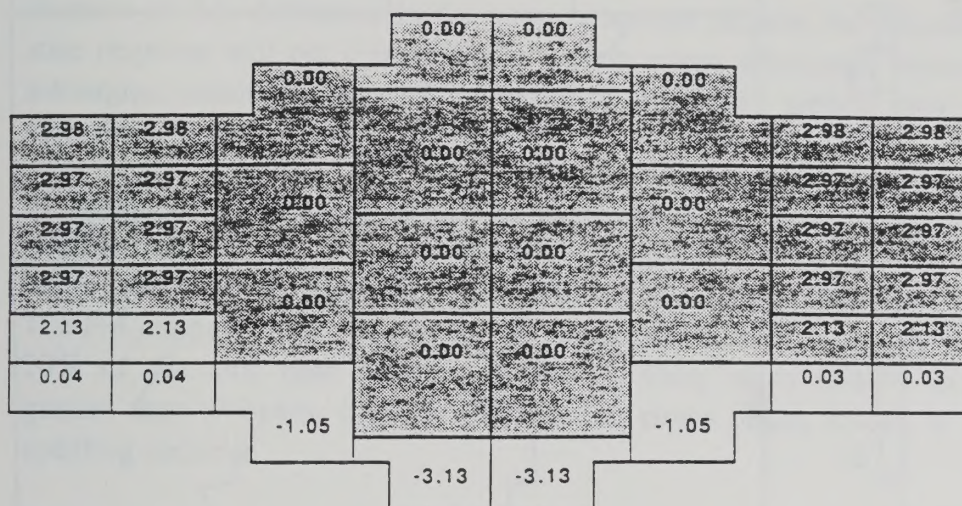
The maximum stress distribution along the elevation of the tower shaft is shown in Figure 5.14 where it may be seen that at roadway level, transverse response contributes significantly to the tower shaft stress.

c. Capacity of the Tower Under Differential Support Movement

Data in the long period range of ground motion lacks the degree of accuracy of the short period range. The topics of spatial and temporal variability of strong ground motion have only recently been addressed through the installation of several strong motion dense arrays in California and Taiwan. All these installations are located on soil sites. The knowledge base of the spatial variability of ground motions at rock sites is still very limited. It is therefore important to carry out sensitivity studies to ensure an adequate capacity available in the tower to tolerate differential support movements.

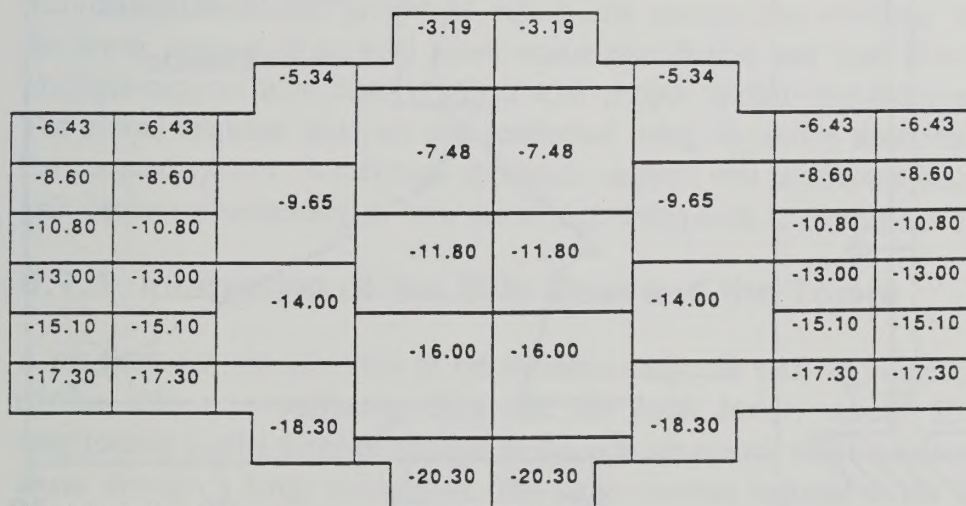
In the quasi-static deformed configuration of the structure, the longitudinal relative displacement component between an anchorage and the adjacent main tower appears to have the most significant effect on the entire structure. Between the anchorage and the top of the main tower, the main cables provide the only longitudinal restraint. The tower is highly flexible in the longitudinal direction as compared to the stiffness provided by the cable tension. By specifying a unit longitudinal displacement at the anchorage while fixing all other supports, the side span cable tension will be relaxed. The unbalanced horizontal cable force at the top of the tower will be transferred to the tower and result in the bending of

**** West Shaft ****



 Rivet failed in shear
or Base plate uplifted

**** East Shaft ****



* unit in ksi

Fig. 5.13: Stress Distribution at the Tower Base
Full Transverse Displacement

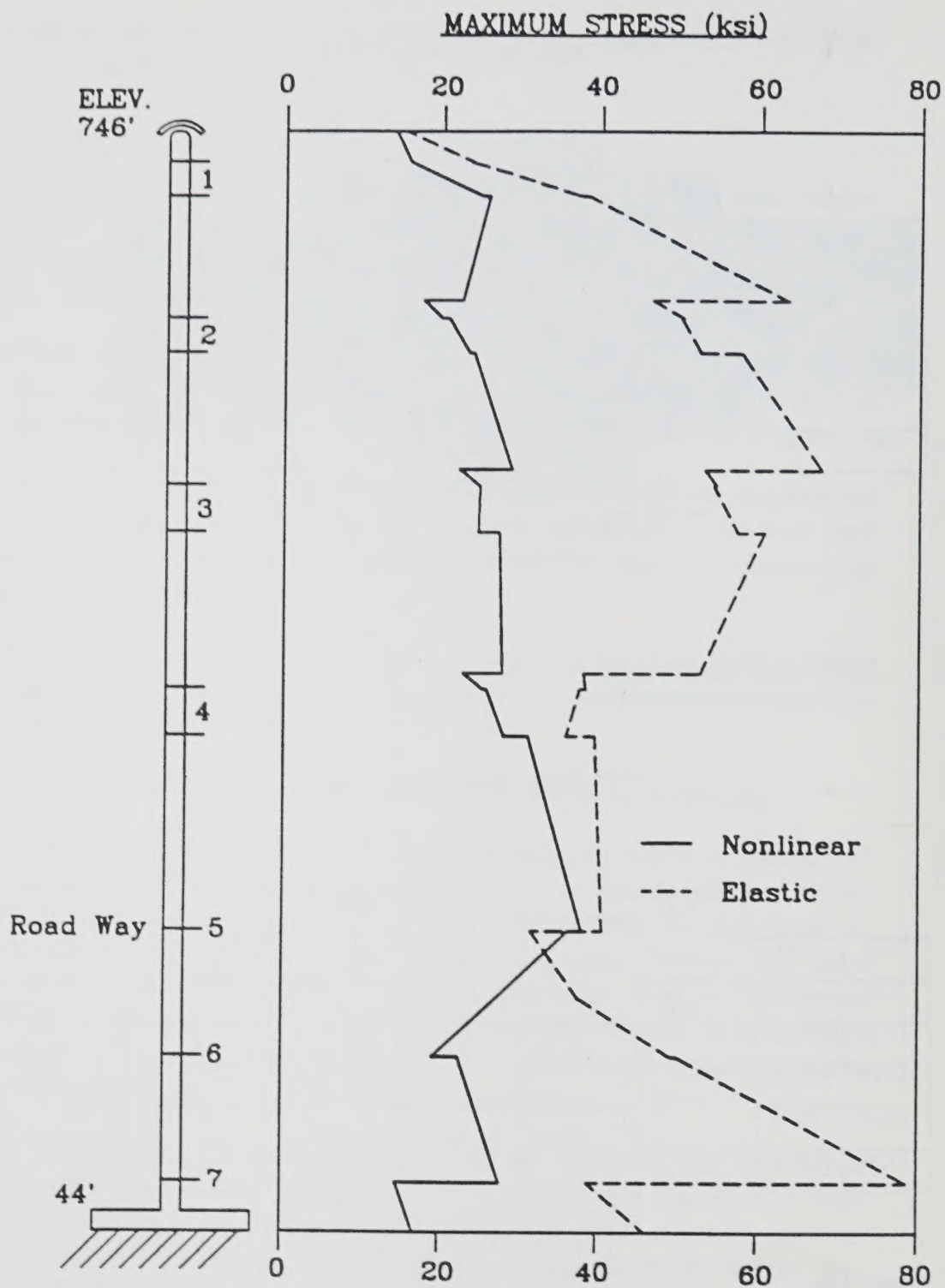


Fig. 5.14: Maximum Transverse Tower Stresses

the tower toward the channel. The displacement at the top of the tower is about the same as the anchorage.

Because of this difference in the tower response pattern, the maximum quasi-state response will not coincide with the maximum vibrational response. In the subsequent sensitivity study of the effect of differential support movements, the vibrational response was not included.

In the absence of vibrational effects, the tower structure can tolerate about 3 ft relative movement at the top (i.e. relative longitudinal displacement between the tower and the anchorage) without significant nonlinearity occurring at the base. The $P-\delta$ moment at the base for the 3 ft displacement at the top contributes about 20% of the total base moment developed. Only when relative displacement greater than 3 ft was imposed hold down, rivets began to fail in shear and uplifting occurred.

Based on the results and considering the additional vibrational effect presented above, it may be concluded that the flexible tower structure has sufficient capacity to tolerate relative support movement in the order of 1 ft. between the tower pier and the anchorage.

5.7 CONCLUSIONS

A detailed seismic evaluation of the Golden Gate Bridge was conducted using a theoretically consistent procedure taking into account the nonlinear behavior at the tower base, such as hold down connection failure and base plate uplifting. Multiple-support time history inputs with proper spatial variability were used. The global response analysis was conducted using the direct time integration of the coupled system. A response spectrum analysis was conducted beforehand to gain better understanding of how the structure responds to earthquake inputs.

5.7.1 Interaction of the Side Span and the Tower

A predominant characteristic of the dynamic response was the coupled vibration of the side span stiffening truss and the main tower. About 80% of the longitudinal inertia force developed in the side span deck will be delivered to the tower through a hinge connection. The large moment induced at the tower base was mostly contributed by the shear force transferred from the side span through the hinge. Since the seismic coefficient used in the original design was only 0.05g (compared to a spectral acceleration of 0.75g for the dominant mode with a period of 1.86 seconds), it is unlikely that the hinge connection can sustain such a high force. If the hinge connection between the side span and the tower failed, there will be a high possibility for impacting to occur between the

suspended structure and the tower. The immediate damage may be expansion joint failure and other serviceability problems. In addition, the potential consequence of additional higher modes of tower vibration being excited by the impact needs further investigation.

5.7.2 Concrete Pylons

Another apparent vulnerability is the shear strength of the concrete pylon. The results of this investigation show that at a slightly higher than the lower level earthquake event (with an average return period of 72 years), shear failure in the concrete pylon may occur. Since the main cables are anchored to the pylon concrete through tie-down cables, damage to the concrete pylon may have a significant effect on the overall performance of the main bridge.

5.7.3 Main Towers

Under the assumption that the side-span-to-main-tower hinge connection remains intact, the base connection of the main tower will experience some damage at an earthquake level slightly above the lower level event. This damage may be of local nature such as the hold down rivet connection shear failure. As earthquake excitation level increases, the extent of damage at the base increases and the entire base section as a whole softens. At the upper level earthquake with an average return period of 475 years, the rotational ductility demand at the base is estimated at about 2.2. About half of the base section under each shaft is uplifted under the rocking response of the tower. However, because of the "softening" effects, stresses at the base section web plate and the bearing stress on the concrete pier will be reduced. These are the beneficial effects of some rocking and uplifting response of the tower.

At the upper portion of the tower shaft, significantly higher stresses are induced in the web plate under the maximum credible earthquake. These stress levels are sufficiently high to cause local buckling and tensile yielding in the web plate. Further studies are necessary to assess more precisely their responses under more frequent earthquakes. It can be estimated, however, that potential problems, such as local buckling, exist under the 475 year (upper level) earthquake event. Further study is also required to assess the response of the tower if the side span is disconnected longitudinally from the tower structure.

5.7.4 Side Span Stiffening Truss

High tensile and compressive stresses, computed in the stiffening stress chords of the side span stiffening truss, are probably related to the coupling or interaction between the side span and the tower. Stresses in the main span

stiffening truss were typically low. Since the top chords directly carry the floor system, a local damage to the chord may directly hinder vehicular traffic. In the current configuration of the bridge, strengthening of the stiffening truss chords will be required.

6. APPROACH STRUCTURES EVALUATION

The approach structures of the Golden Gate Bridge consist of the San Francisco Viaduct, the Fort Point Arch, San Francisco Pylons S-1 and S-2, the Marin Anchorage Housing and Pylon, and the Marin Viaduct. Together, these structures support about one half mile of elevated roadway. These approach structures connect the main suspension bridge with the surface roads to the south (in San Francisco) and to the north (in Marin County). The different parts of the approach structures are of structural steel and reinforced concrete construction. They are founded directly on rock.

6.1 SCOPE AND GENERAL APPROACH

The seismic evaluation of the approach structures was conducted as a series of studies of both the local and global response of the various structures to earthquake ground motion, consisting of the following basic components:

1. Detailed review of contract drawings and shop drawings, as well as limited on-site evaluations, to determine the basic load-resisting systems, the details of the articulation, the construction details, the levels of redundancy and ductility, the nature of the existing seismic restrainers, and the overall condition of the existing structures.
2. Development of detailed three-dimensional computer models for use in structural response analysis. These models were developed with particular attention to their sufficiency to capture the dominant seismic behavior characteristics of each component structure.
3. Structural analysis of these computer models to determine their global and local response to self-weight alone, and to earthquake ground motions for the upper level and maximum credible events described in Chapter 3. These analyses provided estimates of both the force response and the displacement response demands on the structural subsystems and components of each structure in its current configuration.
4. Evaluation of the different structural components to determine their force and displacement capacities. These capacities were determined using the acceptance criteria described in Chapter 4.
5. Vulnerability evaluation, consisting of a comparison of the force and displacement demands (from 3 above) with the force and displacement capacities (from 4 above) to determine the safety of the structures, and to

determine the maximum peak ground acceleration each structure can resist and still meet the acceptance criteria.

6.1.1 Structural Modeling

Detailed computer models were developed for use in the structural response analysis of each of the approach structures. These models were developed with particular concern for their ability to capture the dominant seismic behavior characteristics of each component structure. An additional criterion was that they can be compared with the calculated performance of the structure with modern design criteria. The goal was not to evaluate the true nonlinear behavior of the structures under the given ground motion, but instead to determine whether or not their seismic resistance meets the general requirements of today's seismic engineering practice, and to provide a basis for determining which subsystems and/or components of the structures are in need of seismic upgrade.

Three-dimensional linear models of each entire structure were developed in order to capture their overall response as a structural system to the earthquake ground motion. Local models of a typical supporting frame of the south viaduct was also developed to determine the nonlinear response characteristics of these frames.

a. Global Three-dimensional Models

The global three-dimensional computer models of the approach structures were generated for static and dynamic analysis with the commercially available structural analysis program SAP90 [31]. This program was developed for the linear static and dynamic analysis of general structural systems idealized as a system of finite elements. It has extensive capabilities for response spectrum and time-history analysis under earthquake ground motion, and for checking the results of the analyses against modern design criteria. The following modeling conventions were used for the steel and concrete structures:

Steel Structures

The steel structures of the San Francisco Viaduct, the Fort Point Arch, and the Marin Viaduct are of girder, truss, and arch construction which support steel orthotropic deck roadways and concrete sidewalks. The true geometry of the structures was modeled, including horizontal and vertical curvature. Structural components considered in the global models include the members of the supporting frames and superstructures, the bearings supporting the superstructures on the supporting frames or foundations, and the expansion joints in the trusses. The deck and sidewalks are provided with expansion joints

approximately every 50 ft and therefore do not provide global stiffness or strength to the structures. Only their mass contributions were included in the computer models.

Truss members were modeled as three-dimensional frame elements. All primary and secondary structural framing was explicitly included in the structural models. In order to gain a better understanding of the primary contributions to the seismic resistance of the structures, several computer models were studied based on different idealizations of these members. In one idealization the frame elements were formulated considering axial distortions only (for idealized truss behavior), in the other the formulation included axial, bending, shear, and lacing distortions (for idealized frame behavior). In the latter case, consideration of the lacing distortions is particularly important in the evaluation of bending stresses in the members. The overall dynamics (i.e. frequencies and mode shapes) and the seismic behavior of the structural system, however, are adequately captured using the simpler truss idealization.

The riveted plate girders used for the short spans of the south viaduct, as well as the floor beams supporting the roadway, were in all cases modeled on the basis of idealized frame behavior.

Bearings are used in all the steel approach structures to support the superstructure on their supporting frames or, for the arch, directly on their foundations. Both the geometry and articulation of these bearings influence the overall behavior of the structure. Their geometry is modeled using short frame elements; their articulation is modeled using member end releases and global constraints.

Restrainers were installed across the bearings of the approach structures in a seismic upgrade performed in 1982. These restrainers consist of threaded bars connected to plates bolted to the members on either side of the bearings. The structures were modeled both with the joints open and the restrainers acting (the tension model) and with the joints closed and the restrainers slack (the compression model). The restrainers were modeled explicitly with short truss elements. For the tension model, the truss elements were modeled with the actual stiffness of the restrainer bar. For the compression model, the restrainers were modeled with a high stiffness representing joint contact in the direction of the restrainers. The results of the tension and compression model analyses were taken as providing bounds on actual structural response. These assumptions are in compliance with current Caltrans seismic analysis guidelines for highway bridges.

Reinforced Concrete Structures

The reinforced concrete structures of the Pylons and Marin Anchorage Housing are of monolithic reinforced concrete panel and frame construction, with reinforced concrete slab roadways and sidewalks. Structural components considered in the global models include the exterior shear walls with integrated columns and beams, and for the Marin Anchorage Housing the internal moment-resisting framing supporting the roadway slab. The internal framing was modeled using three-dimensional frame elements representing the gross uncracked properties of the framing. The exterior shear walls were modeled using shell finite elements. Two-dimensional models of interior frames were used for detailed analysis.

b. Nonlinear Analysis Models

Nonlinear analyses of a typical supporting frame for the south viaduct were performed in order to evaluate the response characteristics and system ductility capacities of these subsystems. The intent of the analyses was to provide a rational estimate of the risk to the structure of allowing members to buckle or yield, by evaluating the post-yield and post-buckling behavior of the frames. This method provides a basis for setting the allowable ductility demands on members.

The analyses were performed on a simple planar models of one face of a supporting frame. The nonlinear analyses were limited to static loading-to-collapse evaluations under monotonically increasing imposed forces and displacements.

These analyses demonstrated that the risk to the structure of the failure of diagonal elements of the supporting frames is somewhat less than the risk due to failure of vertical elements in the supporting frames. After buckling of a vertical element, the structure becomes unstable. After buckling of a diagonal element, there is some reserve capacity, up to about double the load at which the diagonal buckles. Due to the low buckling load of the diagonals, the total lateral load capacity of the frame, as governed by failure of diagonals, is somewhat lower than that predicted by the linear global analyses.

6.1.2 Structural Response Analysis

Structural analyses of the global three-dimensional computer models were undertaken to determine their global and local response to self-weight alone, and to earthquake ground motions for the upper level and maximum credible events described in Chapter 3. These analyses provided estimates of both the force

response and the displacement response demands on the structural subsystems and components of each structure in its current configuration.

Analysis of the approaches including the effects of multiple-support excitation was not undertaken. The rigid-base excitation analysis was deemed sufficient, and the multiple-support excitation analysis was deemed unnecessary for these particular structures which are externally statically determinate. The following reasoning was used: under multiple-support excitation, total structure response consists of the sum of dynamic and quasi-static components. For the computed internal forces and displacements, the following facts are known with respect to the values of the dynamic and quasi-static components:

Component	Force response	Displ response
<i>Dynamic</i>	non-zero	non-zero
<i>Quasi-static</i>	zero	non-zero

The quasi-static force response is known to be zero since the structures are externally statically determinate. Therefore, a multiple-support excitation analysis is unnecessary for the evaluation of structure forces under earthquake excitation. The quasi-static displacement response is non-zero, however. Therefore, the dynamic component of displacements will under-estimate the maximum displacements, and accurate evaluation of displacements would require the multiple-support excitation analysis. However, the rigid-base excitation analysis was sufficient to determine that the displacement capacities of the bearings and joints currently in the structure are insufficient.

Response spectrum analyses were used for computation of structural response. The computations were performed initially using eigenvectors as a basis; however a very large number of modes was required in order to account for an adequate percentage of structural mass. In subsequent analyses, Ritz vectors [32] were used as a basis, which provided an adequate analyses with fewer modes.

It is generally recognized that for structures whose fundamental periods fall in the range found for the approach structures, the displacement response of the nonlinear yielding structure is approximately the same as that of the linear structure. However, the internal forces will be quite different. This was recognized in this evaluation by employing the concept of ductility ratios for the elements.

6.1.3 Vulnerability Evaluation

The vulnerability evaluations of the approach structures consisting of a comparison of the force and displacement demands, determined by the structural

response evaluation, with the force and displacement capacities of the structural components, determined by application of the acceptance criteria discussed in Chapter 4. This allows the determination of the safety of the structures under a particular ground motion, and the determination of the maximum peak ground acceleration each structure can resist and still meet the acceptance criteria.

a. Truss Members

The procedure used in this evaluation of the truss members of the steel structures consisted of several steps as described below:

1. For the combination of dead load and seismic ground motion for each level earthquake, determine the utilization factor (demand/capacity) based on the AISC/LRFD specifications [26].
2. Classify members according to member categories 1 through 3 as described in Chapter 4.
3. Create a database for each structure, containing a record for each member. For each member, the database fields contained the following data:
 - Member type classification (category 1, 2, or 3).
 - Slenderness ratios KL/r .
 - Local buckling ratios b/t .
 - Utilization factors under dead load and various earthquake input.
4. Screen the database for members that do not satisfy either local or global buckling criteria, and for members with utilization factor greater than 1.0.
5. For members that do not satisfy the b/t ratio criteria, the b/t ratio is converted to an equivalent Kl/r ratio using theory provided in [29]:
 - For stiffened elements $(Kl/r)_{eq} = 1.65(b/t)$
 - For unstiffened elements $(Kl/r)_{eq} = 5.06(b/t)$

These members are then checked against the global buckling criteria using the equivalent Kl/r .

The result of this screening is a table of members which do not satisfy the truss member criteria discussed in Chapter 4.

b. Other Structural Components

All other structural components were checked for compliance by direct comparison with the criteria discussed in Chapter 4. the following points can be made:

1. The compliance of the bearing shoes with respect to force requirements was governed by shear and tension in the bolts used for attachment to the supports.
2. The compliance of the gusseted connections in the trusses was governed by shear in the riveted connections.
3. The compliance of the foundations was governed by the capacity of the anchor bolts. The total uplift capacity of the foundation pads was also deficient, but less so than the bolts.
4. The compliance of the expansion joints with respect to displacement capacities was in general far below requirements.
5. The compliance of the threaded bar restrainers across the joints was governed by their connection details and by the non-ductile nature of their behavior.
6. The compliance of the reinforced concrete components was governed by basic strength requirements, plus the non-current detailing used which limits the available ductility.

6.2 SAN FRANCISCO VIADUCT EVALUATION

The San Francisco approach viaduct is on a horizontally-curved alignment and consists of three steel girder spans and three steel truss spans. This structure spans from the vicinity of the toll plaza to the south to the reinforced concrete San Francisco pylon S2 to the north. Its total length is about 700 ft. A plan and elevation of the viaduct are shown in Fig. 6.1. A cross section through a typical support frame is shown in Fig. 6.2. A plot of the three-dimension mathematical model used for structural analysis is shown in Fig. 6.3.

Members not in compliance with the design criteria and therefore in need of seismic upgrade are illustrated in Fig. 6.4. Members in compliance with the design criteria and therefore not in need of upgrade are illustrated in Fig. 6.5. The range of maximum peak ground accelerations which can be resisted within the acceptance criteria by each component type in the viaduct is shown in Fig. 6.6.

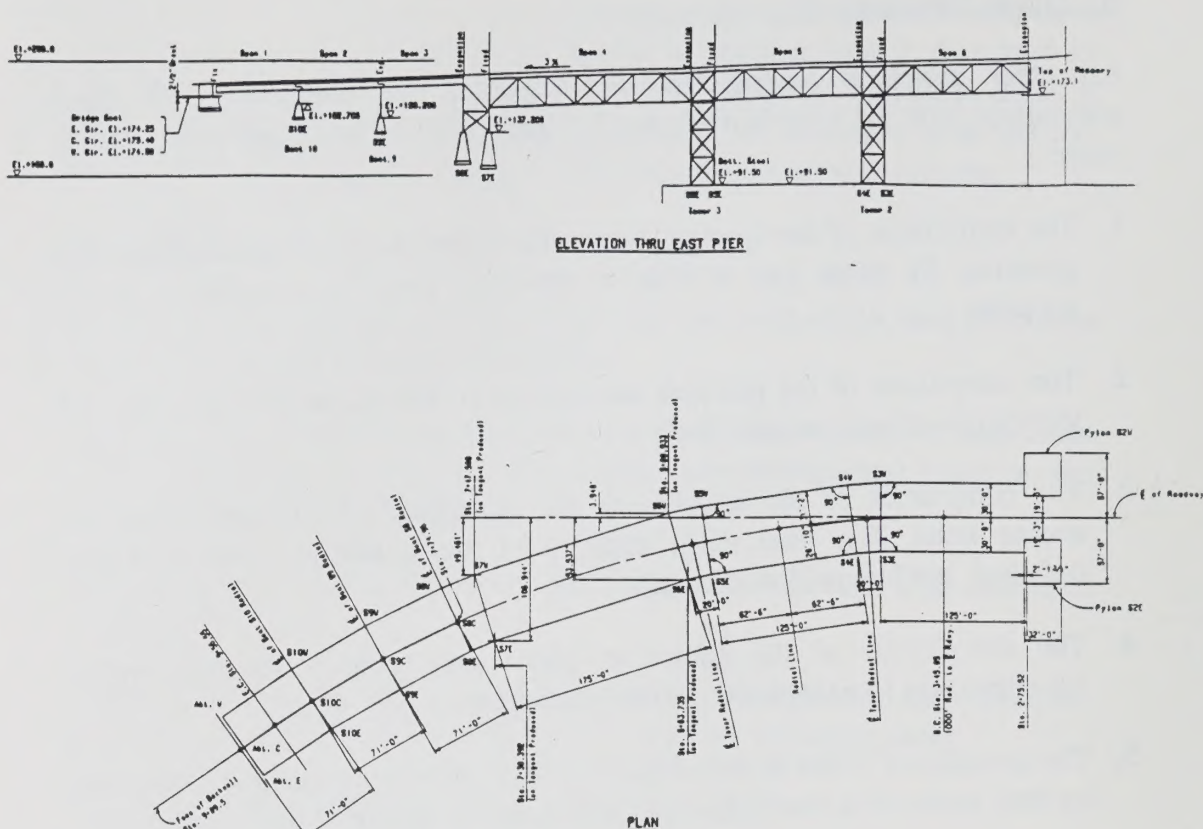


Fig. 6.1: San Francisco Viaduct Plan and Elevation

Based on this evaluation, Table 6.1 summarizes the strength improvements that are required for the components of the San Francisco viaduct to meet the seismic performance criteria under maximum credible earthquake. In addition, expansion joint movement capacities must be increased by at least a factor of three to limit risk of loss of service.

Table 6.1: Upgrade Requirements for San Francisco Viaduct

Component	Maximum	Minimum
<i>Footing Uplift</i>	15.50	8.86
<i>Footing Shear</i>	5.64	2.95
<i>Member 1 (Towers)</i>	3.65	2.58
<i>Member 1 (Girders)</i>	2.21	1.35
<i>Bearing Shoes</i>	10.33	4.77
<i>Gusset Plates</i>	6.89	2.95
<i>Member 1 (Trusses)</i>	1.24	1.17
<i>Member 2</i>	1.32	1.19

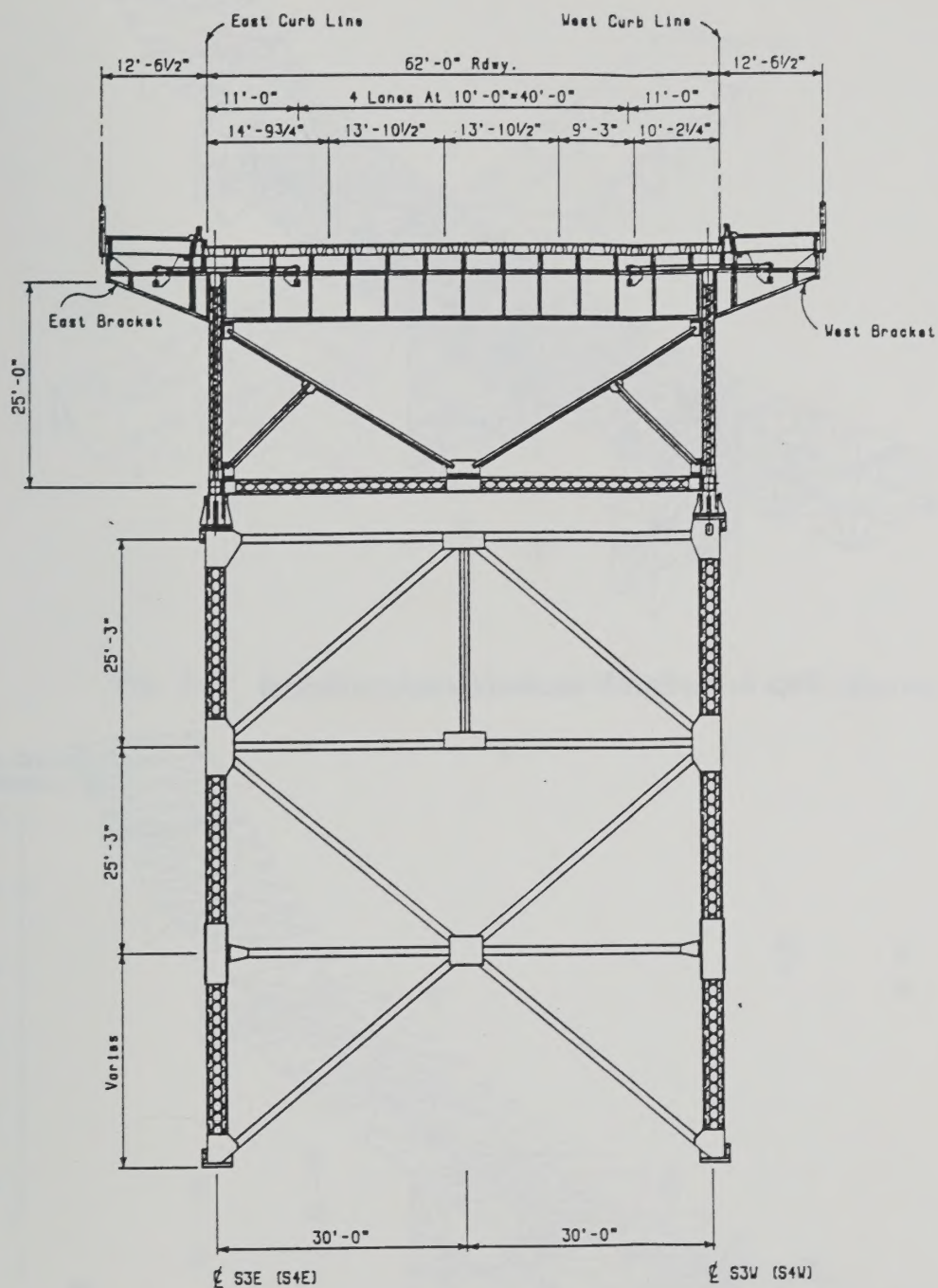


Fig. 6.2: San Francisco Viaduct Cross Section

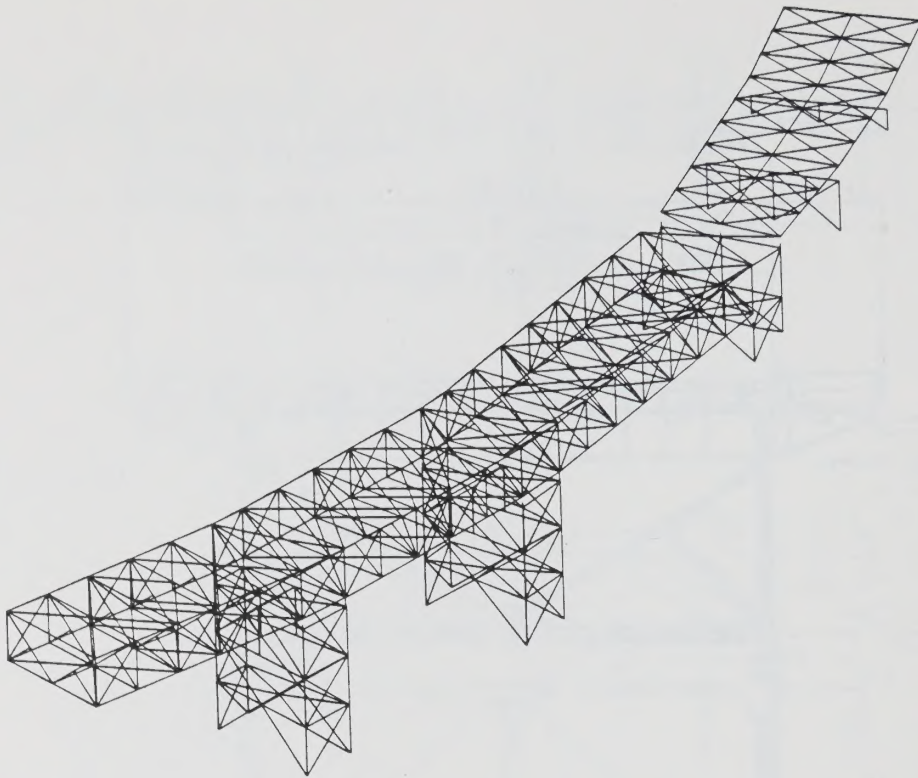


Fig. 6.3: San Francisco Viaduct Model

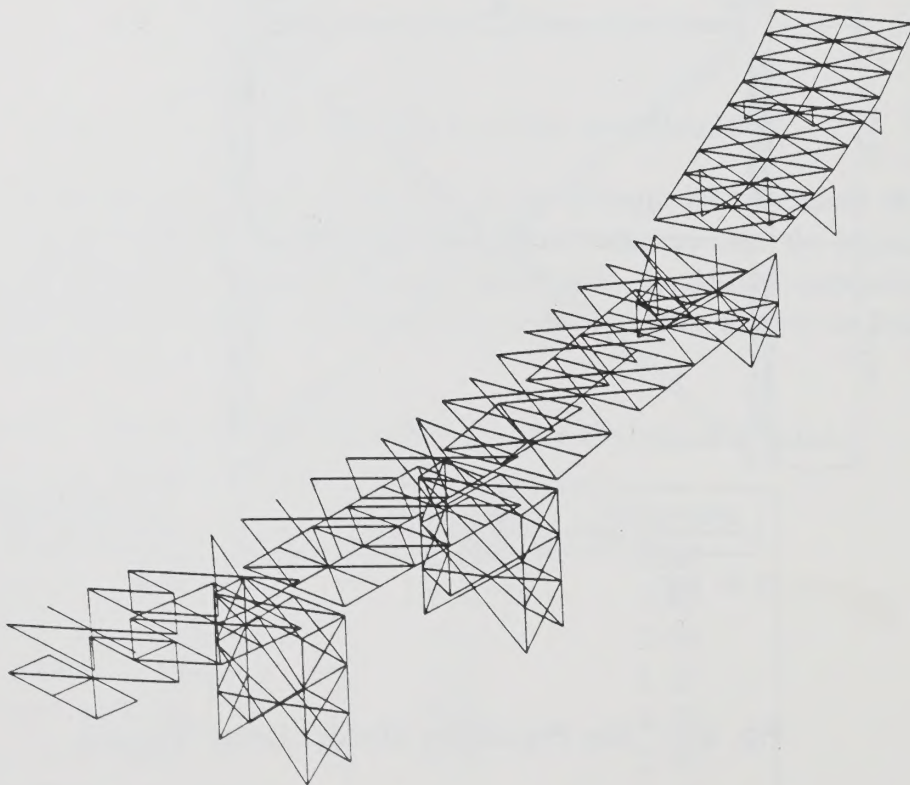


Fig. 6.4: San Francisco Viaduct Members Not In Compliance

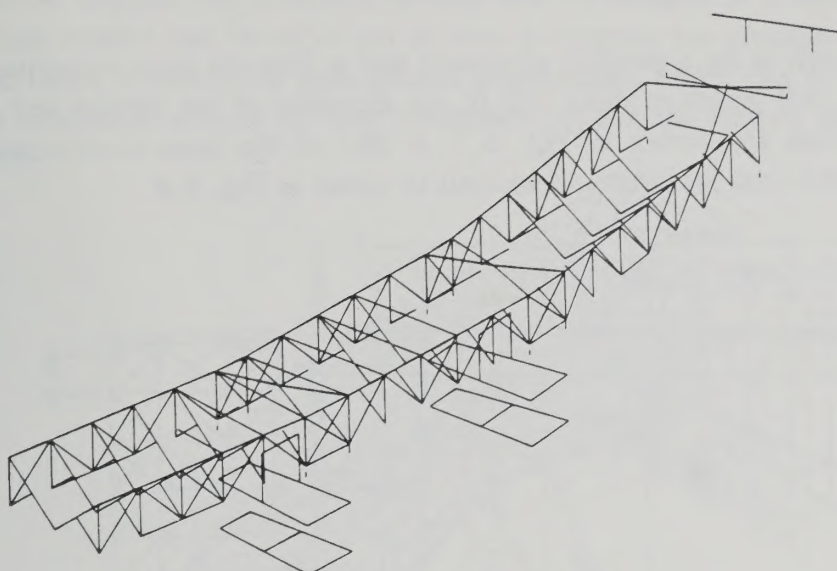


Fig. 6.5: San Francisco Viaduct Members In Compliance

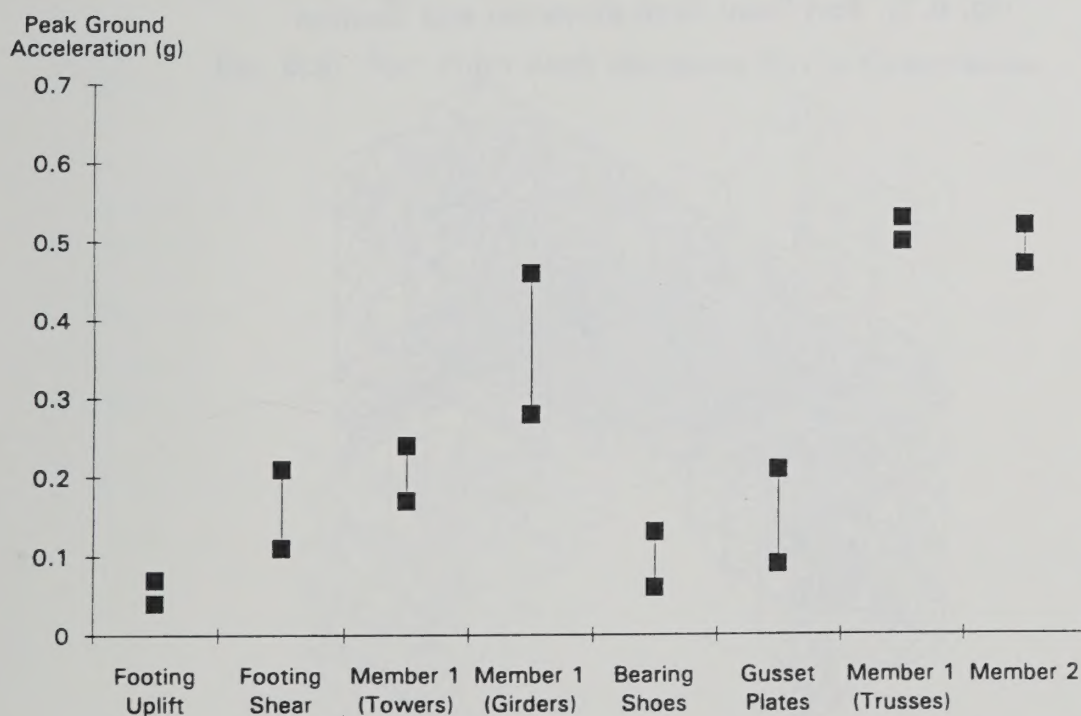


Fig. 6.6: San Francisco Viaduct Component Capabilities

6.3 FORT POINT ARCH EVALUATION

The Fort Point arch is on a straight alignment and a four-rib arch supporting floor beams. Its total length is about 320 ft. An elevation of the viaduct and a typical cross section are shown in Fig. 6.7. A plot of the three-dimensional mathematical model used for structural analysis is shown in Fig. 6.8.

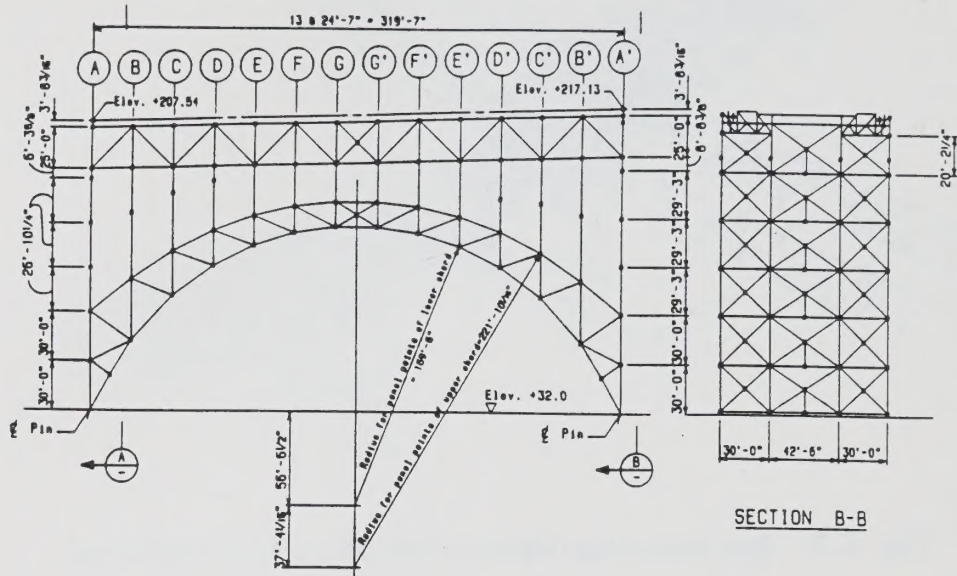


Fig. 6.7: Fort Point Arch Elevation and Section

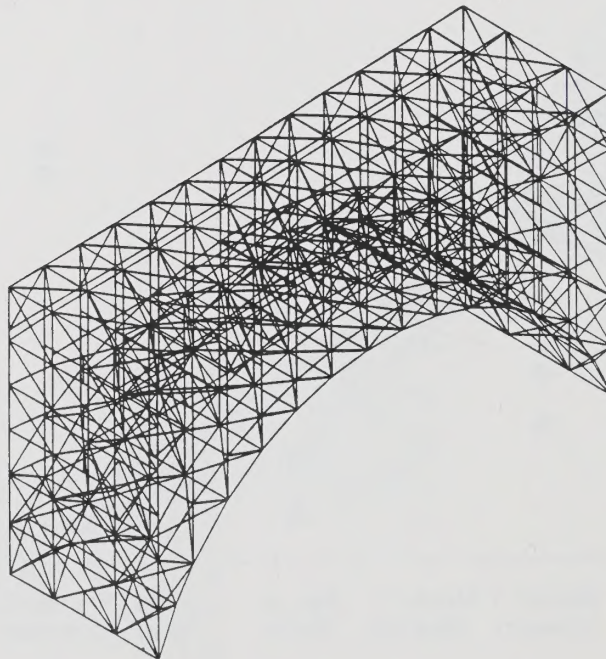


Fig. 6.8: Fort Point Arch Model

Members not in compliance with the design criteria and therefore in need of seismic upgrade are illustrated in Fig. 6.11. Members in compliance with the design criteria and therefore not in need of upgrade are illustrated in Fig. 6.12. The range of maximum peak ground accelerations which can be resisted within the acceptance criteria by each component type in the viaduct is shown in Fig. 6.13.

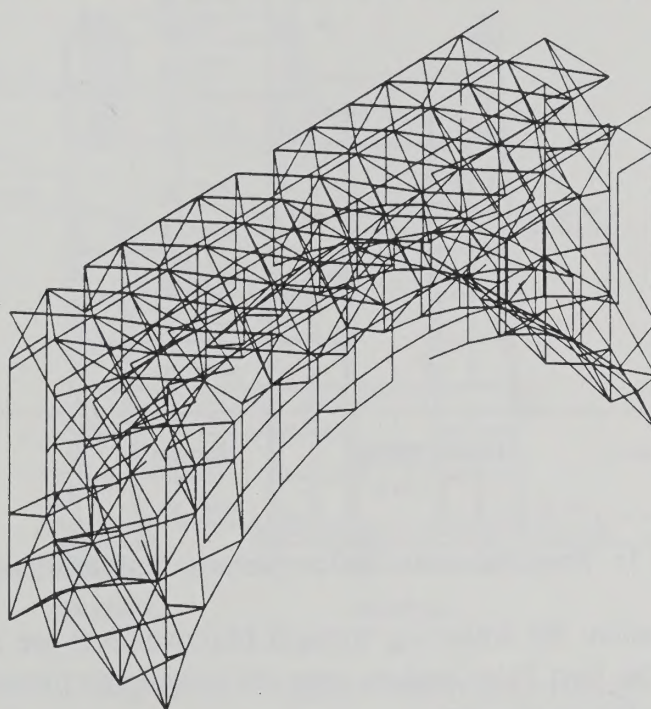


Fig. 6.9: Fort Point Arch Members Not In Compliance

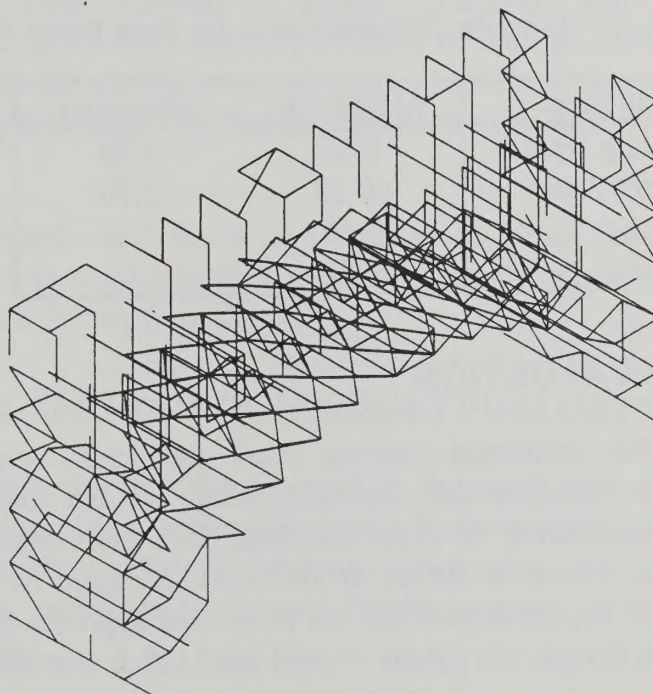


Fig. 6.10: Fort Point Arch Members In Compliance

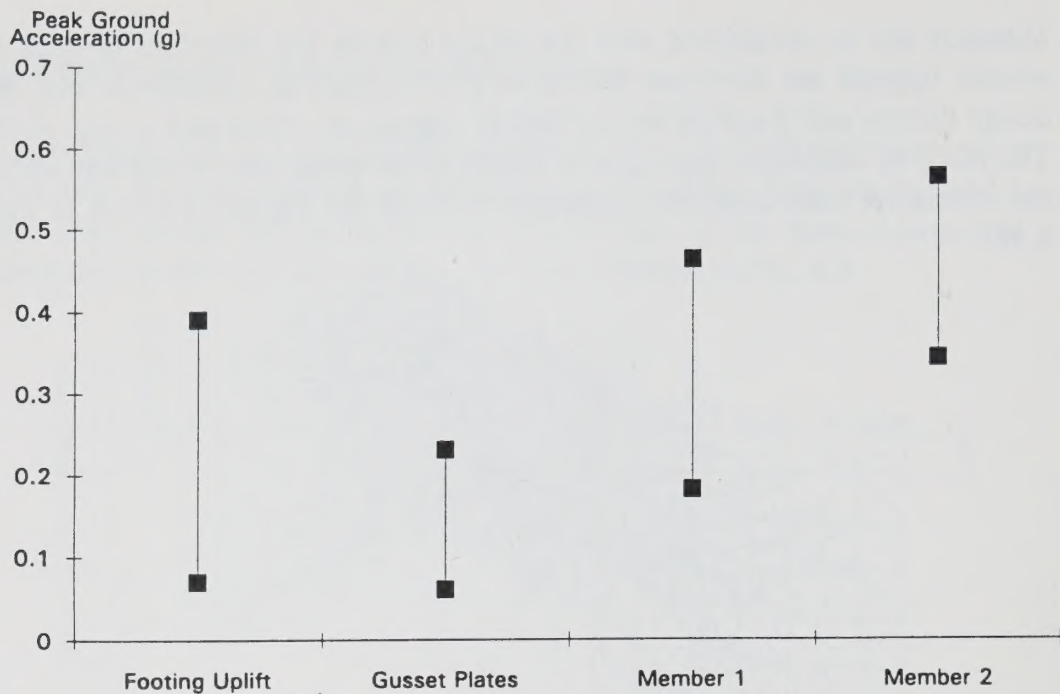


Fig. 6.11: Fort Point Arch Component Capabilities

Based on this evaluation, the following strength improvements are required for the components of the Fort Point arch to meet the seismic performance criteria under maximum credible earthquake:

Table. 6.2: Upgrade Requirements for Fort Point Arch

Component	Maximum	Minimum
Footing Uplift	8.86	1.59
Gusset Plates	10.33	2.70
Member 1	3.44	1.35
Member 2	1.82	1.11

6.4 PYLONS EVALUATION

Pylons of monolithic reinforced concrete panel and frame construction are located at the extreme ends of the suspension bridge and the south end of the arch span. At the south end of the suspension span, the pylons support the end of the 1125 ft side span, as well as the end of the 320 ft arch span. At the south end of the arch span, the pylons support the end of the last approach truss span. At the north end of the bridge, the pylons support the 1125 ft suspension span and

are a part of the Marin anchorage housing. These pylons are up to 250 ft high. An elevation of a typical pylon is shown in Fig. 6.12.

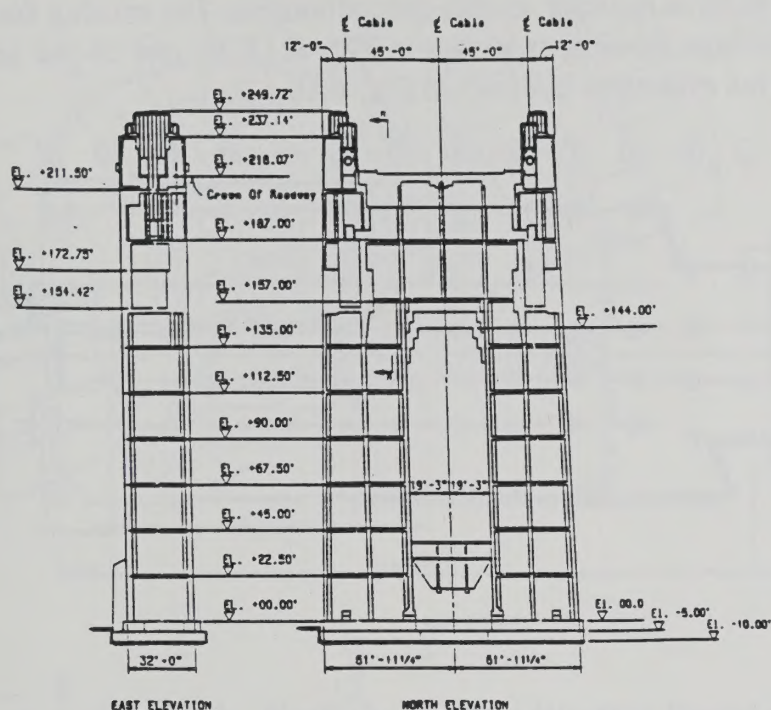


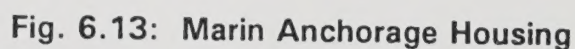
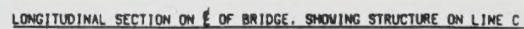
Fig. 6.12: Typical Pylon Elevations

These pylons were modeled as planar structures with frame elements representing the gross cross section properties of the box shafts. These models were included in the global three-dimensional models of the approach viaducts and the suspension bridge to determine their capacity demands as parts of the system.

Base shear demands were found to be one to six times greater than nominal capacities. Base moment demands were significantly greater than the tension capacities of their foundations will resist.

6.5 MARIN ANCHORAGE HOUSING EVALUATION

The Marin Anchorage Housing, located at the north end of the suspension bridge, is a reinforced concrete panel and frame construction similar to the pylons. This structure, about 320 ft long, 120 ft wide, and up to 100 ft high, houses only the anchorage blocks and framing support of the roadway and is completely closed.



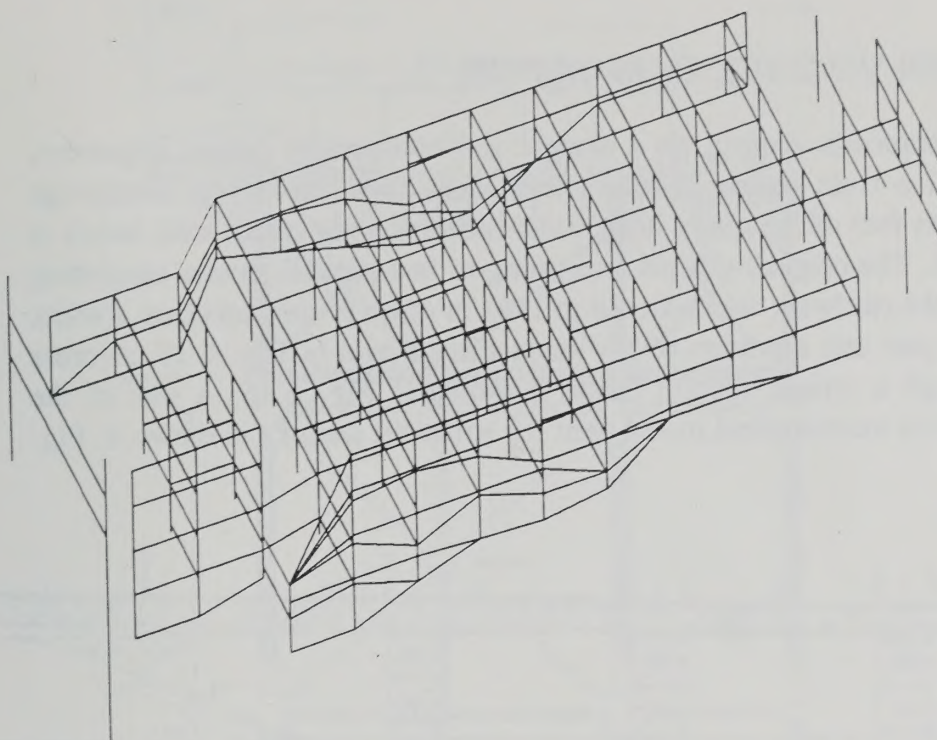


Fig. 6.14: Marin Anchorage Housing Model

Ductility demands on the members of the Marin Anchorage Housing under maximum credible earthquake are summarized in Table 6.3.

Table 6.3: Ductility Demands for Marin Anchorage Housing

Component	Maximum	Minimum
<i>Girders</i>	1.9	6.2
<i>Columns</i>	4.6	7.3
<i>Walls</i>	0.8	8.4

The ductility capacities of these components is very low due to the "low deformation" reinforcing bars used, the lack of shear steel, and the unconfined nature of the concrete. Therefore, the ductility capacities of the elements under earthquake loading is not greater than unity and all components require upgrade in order to meet the criteria.

6.6 MARIN VIADUCT EVALUATION

The Marin Approach viaduct, on a straight and horizontally curved alignment, consists of five truss spans. This structure spans from the Marin Anchorage Housing to the foot of the grade at the north end of the bridge. Its total length is about 1075 ft. The original viaduct, consisting of two parallel trusses supporting each side of the roadway, was widened in 1961 in order to accommodate a wider roadway. A plan and elevation of the viaduct are shown in Fig. 6.15. A cross section through a typical support frame is shown in Fig. 6.16. A plot of the three-dimension mathematical model used for structural analysis is shown in Fig. 6.17.

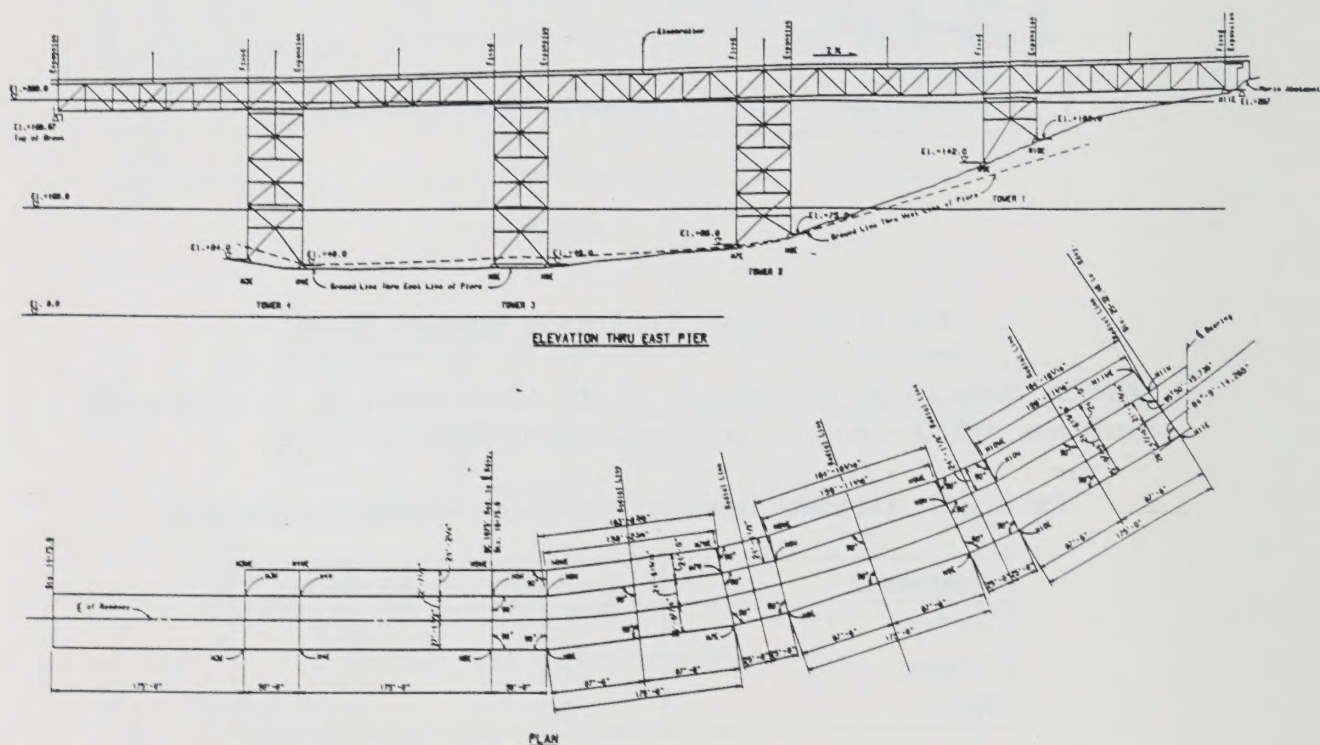
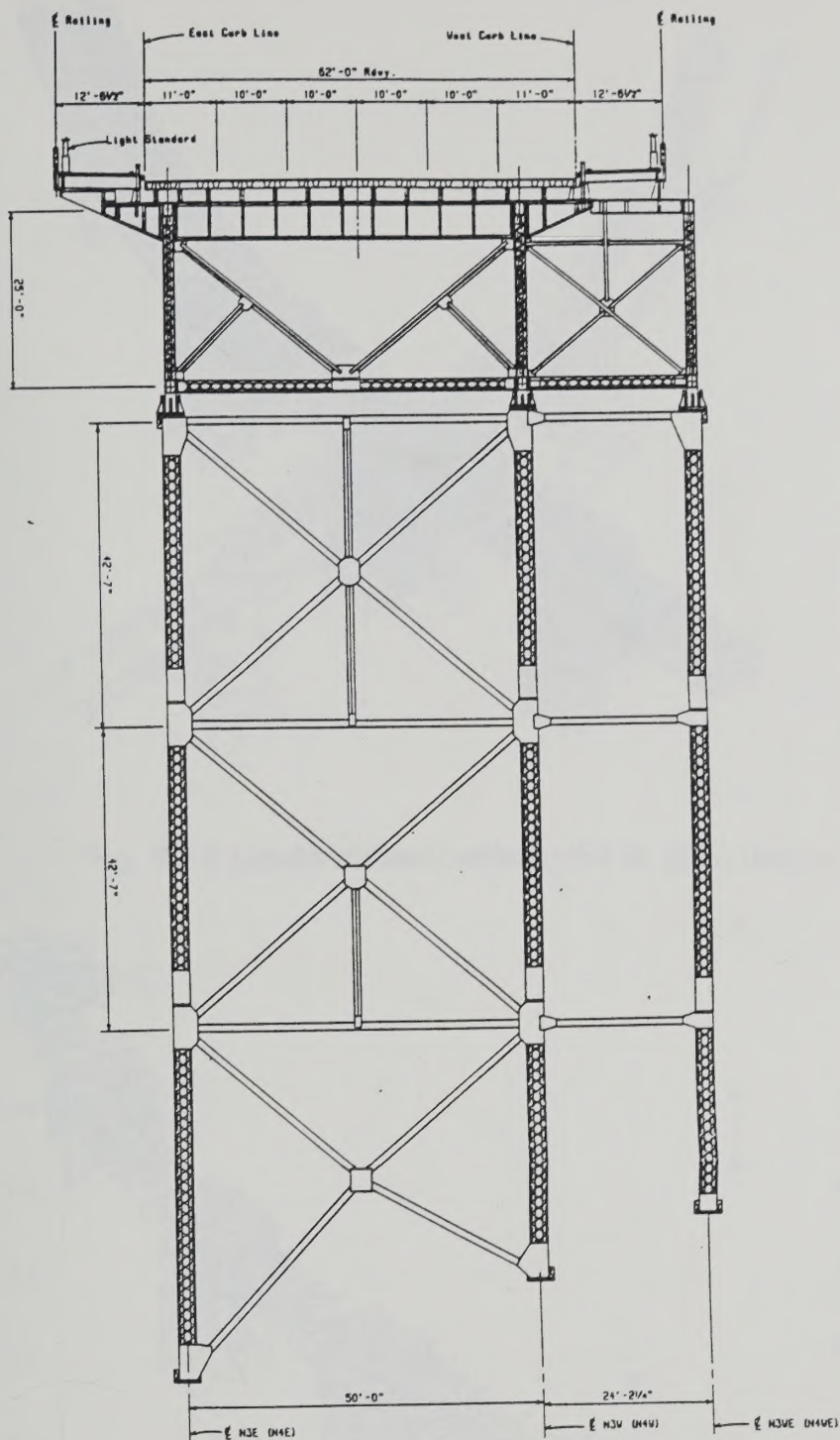


Fig. 6.15: Marin Viaduct Plan and Elevation

Members not in compliance with the design criteria and therefore in need of seismic upgrade are illustrated in Fig. 6.18. Members in compliance with the design criteria and therefore not in need of upgrade are illustrated in Fig. 6.19. The range of maximum peak ground accelerations which can be resisted within the acceptance criteria by each component type in the viaduct is shown in Fig. 6.20.



EXISTING TOWER 4
(LOOKING SOUTH)

Fig. 6.16: Marin Viaduct Cross Section

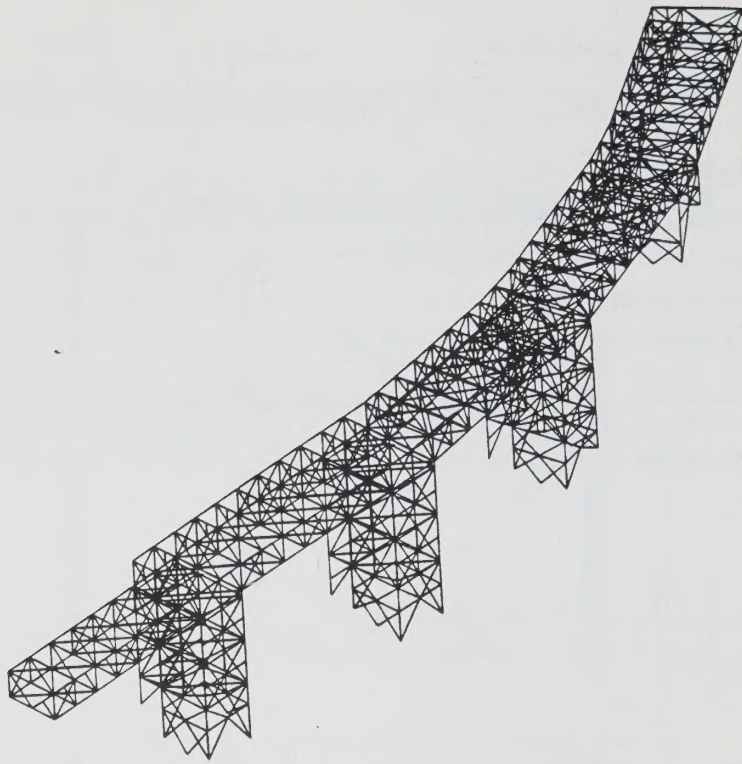


Fig. 6.17: Marin Viaduct Model

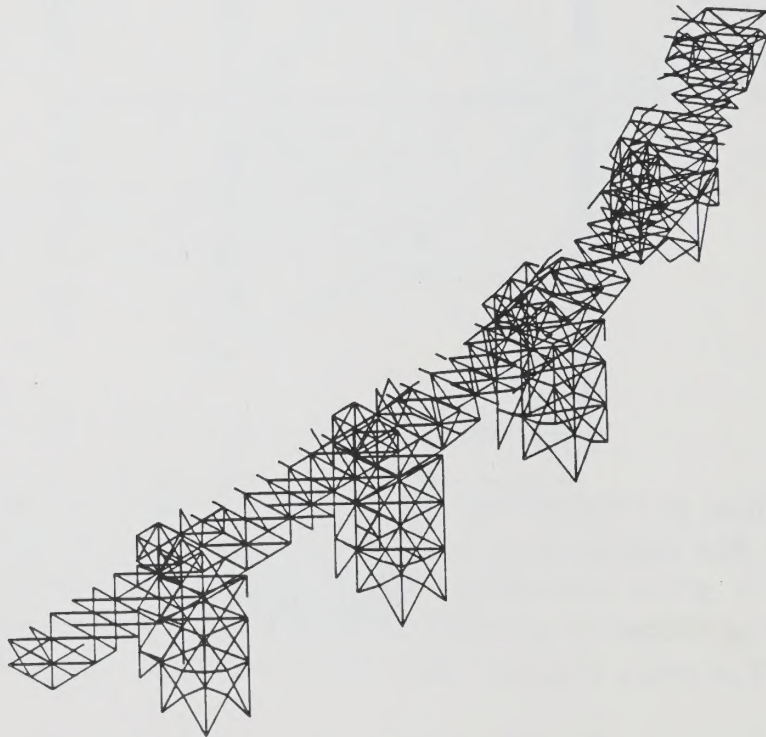


Fig. 6.18: Marin Viaduct Members Not In Compliance

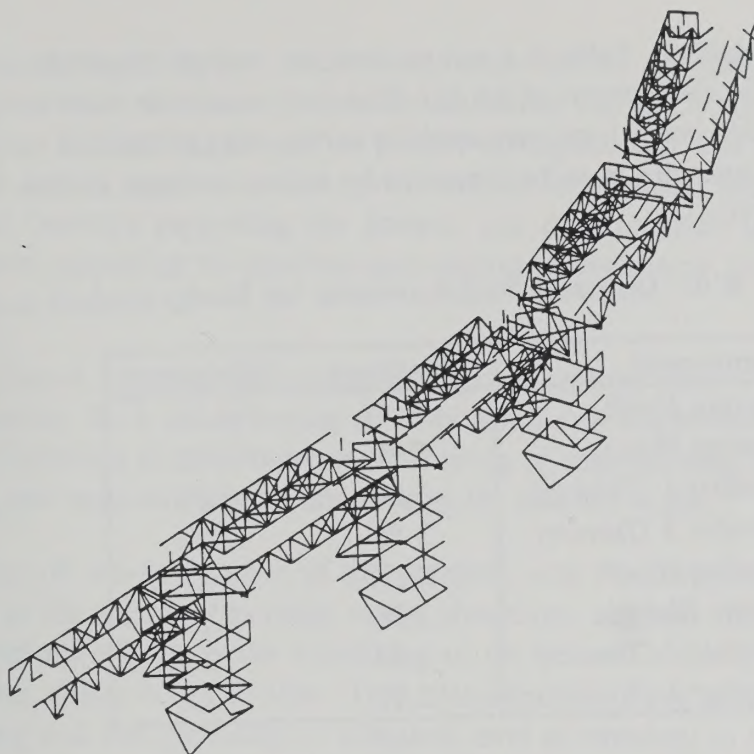


Fig. 6.19: Marin Viaduct Members In Compliance

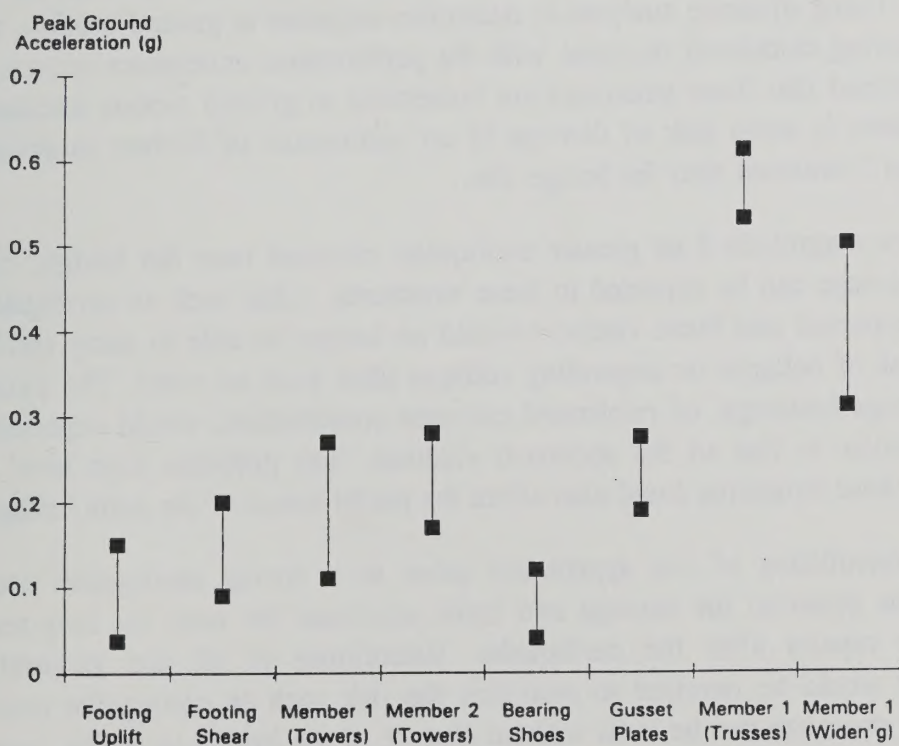


Fig. 6.20: Marin Viaduct Component Capability

Based on this evaluation, Table 6.4 summarizes the strength improvements they are required for the components of the San Francisco viaduct to meet the seismic performance criteria under maximum credible earthquake. In addition, expansion joint movement capacities must be increased by at least a factor of four to limit risk of loss of service.

Table. 6.4: Upgrade Requirements for Marin viaduct

Component	Maximum	Minimum
<i>Footing Uplift</i>	16.53	4.13
<i>Footing Shear</i>	6.89	3.10
<i>Member 1 (Towers)</i>	5.64	2.30
<i>Member 2 (Towers)</i>	3.65	2.21
<i>Bearing Shoes</i>	15.50	5.17
<i>Gusset Plates</i>	3.26	2.25
<i>Member 1 (Trusses)</i>	1.17	1.02
<i>Member 1 (Widening)</i>	2.00	1.24

6.7 SUMMARY

This evaluation of the Golden Gate Bridge approach structures has identified vulnerable components by employing detailed review and evaluation of the structures. Using dynamic analyses to determine response to ground motion, and then comparing calculated response with the performance acceptance criteria, it was determined that these structures are vulnerable to ground motion excitation and that there is some risk of damage in an earthquake of Richter magnitude greater than 7 centered near the bridge site.

In a Richter magnitude 8 or greater earthquake centered near the bridge, high levels of damage can be expected in these structures. After such an earthquake, it can be expected that these viaducts would no longer be able to carry traffic. There is risk of collapse or impending collapse after such an event. The pylons and anchorage housings, of reinforced concrete construction, would experience damage similar to that of the approach viaducts. The probable high level of damage to these structures could also affect the performance of the main bridge.

Structural retrofitting of the approaches prior to a strong earthquake could minimize the potential for damage and could eliminate the need for long-term closure for repairs after the earthquake. Retrofitting of all the vulnerable components would be required to minimize the risk such an earthquake poses. The retrofit measures can be built without closure of the bridge to traffic, using methods similar to those used during the recent deck replacement project.

The steel approach viaducts need to be made two to six times stronger with respect to seismic resistance to provide confidence that the bridge would remain open after a Richter magnitude 7 or greater earthquake centered near the bridge. The most critical components for retrofit are the foundations, supporting frames, structural bearings supporting the trusses, and the expansion joint restrainers. The trusses supporting the roadway also require strengthening, but are somewhat less critical.

The reinforced concrete anchorage housings and pylons need a similar level of strengthening. Such strengthening could be similar to the seismic reinforcement of older buildings in San Francisco, consisting of strengthening the foundations, building new internal framing, and bracing the exterior walls.

The extent of non-compliance of the structure with the acceptance criteria, and the risk to the region of damage to the structures, suggests that in addition to component retrofitting, some retrofitting of the seismic resisting systems of the approaches should be undertaken. This alternative has many advantages in terms of limiting risk and providing an adequate level of certainty in the performance of the structures.

7. SEISMIC UPGRADE ALTERNATIVES

The seismic evaluation reported in the preceding Chapters has found that both the suspension spans and all the approach structures need seismic upgrading in order to safely survive a moderate to great earthquake. Seismic upgrading means that the bridge's components or structural systems must be altered or retrofitted to improve its ability to safely survive seismic shaking. There are many alternative retrofitting methods by which the seismic upgrading may be accomplished. Each alternative should be evaluated for its seismic mitigation effectiveness, its feasibility of retrofitting the member or structural system, its construction schedule, construction cost, and its aesthetic and historic impact on the bridge.

The emphasis of this seismic evaluation was to determine the seismically vulnerable members, joints, and structural systems under anticipated severe seismic events. Evaluating seismic upgrade alternatives in detail and for estimated construction cost was not a part of the study. However, the next step in the process of upgrading the Golden Gate Bridge should be an evaluation of seismic upgrade alternatives. After the completion of the evaluation of the alternatives, the District will have a clearer understanding of what will or will not work. More importantly, the District will have better cost estimates and construction schedules for the seismic upgrading and can better plan for future funding and construction schedules.

Generally, seismic upgrade alternatives can be in two forms: structural modifications or structure response modifications. Structural modifications usually strengthen the members or structural system. They can be further divided into methods such as restraining, bracing, strengthening, replacing, or some combination of these methods. Structure response modifications usually alter the manner in which the structural system responds to the earthquake. They include such items as base isolation, which also can be applied effectively above the base of the structure, energy dissipators or absorbers, active or passive tuned-mass dampers, controlled uplift devices, structural fuses, and seismic lock-up devices. Structure response modification can also be achieved by certain changes in the structure system, such as changing the articulation of structural components or creating composite action by linking together two elements that previously acted independently.

7.1 STRUCTURAL MODIFICATIONS

7.1.1 Restrainers

In the twenty years since the San Fernando earthquake vividly showed the need for seismic upgrading of bridges, most structural modifications have focused on modifying the structure to improve its ability to hold together or to prevent its slipping off its supports. The predominant structural modification has been the addition of restrainers installed at joints and girder supports to hold the structure together and on to the supports. One of the seismic modifications added to the Golden Gate Bridge in the 1980's was the installation of restrainers.

Restrainers, however, are very limited in their ability to protect the structure if other components are more vulnerable to seismic shocks. To be effective, restrainers need to be considered as only one item in a structural system that is being shaken by the seismic action.

For bridge structures that are severely overstressed, as are the approach structures of the Golden Gate Bridge under strong seismic forces, restrainers may not be effective.

7.1.1 Bracing and Strengthening

Adding bracing is one form of strengthening of the structure against seismic action. Bracing reduces the unsupported length of compression members or provides a stiff load path for inertia forces from the structure passed to the ground. Some bracing was added to the Golden Gate Bridge approach structures in the 1970's.

Bracing may have some but limited application to the steel approach structures. The concrete pylons and the Marin Anchorage Housing could be strengthened with steel or concrete bracing constructed inside the hollow spaces of the pylons or housing. This internal steel bracing would add strength to the concrete pylons without distracting from their art-deco design.

Strengthening techniques include adding steel materials such as cover plates to members, replacing rivets with high-strength bolts, and adding concrete and providing better anchorage to piers or foundations. Overstresses in the main towers of the suspension span may be reduced by strengthening in the form of cover plates or other devices to increase steel section area and brace against buckling action. If components are greatly overstressed, strengthening will not work and other methods must be used.

Another form of strengthening is to provide confinement to reinforced concrete components in order to improve ductility. This technique could be applied to the piers, the pylons, and the Marin Anchorage Housing.

7.1.3 Replacement

Replacement may be the only solution for overstressed members, joints or footings. However, replacement requires removal and replacement of pieces of the historic structure and is the most expensive, difficult, and time-consuming method of seismic upgrading. Nonetheless, if the overstresses are so great that replacement is the only method of insuring the seismic safety of the structure, it must be considered as a viable method. Replacement operations have been done to the Golden Gate Bridge a number of times: thousands of rivets have been replaced, as have the suspender cables and the concrete deck on the main spans.

Once the replacement alternative is decided upon as the only viable alternative, the next step in the engineering design process is to develop practical methods of removing and replacing components that contractors can use to insure the safety of the structure while maintaining continuous traffic for vehicles and pedestrians. Temporary supports to relieve the loads must be devised to support the structure while the members or footings are removed and replaced. The temporary supports must support both the dead and live loads and possible wind and temperature forces with a high factor of safety. Most importantly, the operation of unloading the temporary supports and transferring the load back to the replacement member must be controlled so that the proper geometry and distribution of stresses are obtained in the altered structure.

It appears that all of the approach steel towers and footings may require total replacement. If this is verified as the only retrofit measure available, then at least two methods are available. The first method is to replace the steel tower in its exact position. This method would require the trusses supported by the steel towers to be temporarily shored up by falsework while the tower's concrete footings are reconstructed, then dismantled, and the tower then rebuilt back in place. Then the load must be transferred by controlled jacking from the falsework back to the new steel towers and the falsework removed.

Alternatively, a new permanent steel tower could be constructed around the existing tower. Controlled jacking could then transfer the load from the original to the new tower and the original tower can be dismantled. New footings drilled into bedrock to take tension as well as compression loads can be more easily constructed by this method. This method has the advantage of being simple and perhaps lower in cost, but it may change the appearance of the bridge slightly and may require permits for wider footings.

The replacement method is probably not applicable to the suspended spans.

7.2 STRUCTURE RESPONSE MODIFICATION

Modifying the manner in which the structure responds to seismic action is a seismic upgrade alternative that may have potential savings over other methods. Since this method changes the structural response of the structure system, the investigation of the method must be carried out by means of computer simulations. Once its effectiveness is proven, some structural alternatives may be necessary to add the device to the structure. Response modification devices can also develop secondary effects that must be fully evaluated to insure proper function and safety of the structure.

7.2.1 Base Isolation

Base isolation has been applied to buildings and more recently to some bridges. There are several base isolation devices on the market, but all of them require inserting a device into the structural support system to "isolate" the structure from the ground movements. No device can do this perfectly but some attenuation can be achieved that reduces the seismic inertia forces above the isolation unit.

The secondary effects of base isolation are larger displacements in the structure. For bridges, large displacement of the roadway at expansion joints may prove to be a traffic safety hazard. Placing the device at the top of the column supporting the approach trusses may reduce distress in the truss members and control deck displacements to an acceptable level. Base isolation can not be used on the main spans.

7.2.2 Energy Absorbers

Energy absorbers, damping devices and energy dissipaters are exactly what their names imply. By absorbing energy or damping movements the devices redirect the damage-causing energy away from the structure. The devices also must be inserted into or added onto the structure. Secondary effects are heat buildup, the need to install many devices to effect a reduction in seismic forces as the devices can cause local fires at the points of attachments. These devices can not be used on the main spans.

Tuned mass dampers have been used on buildings and recently on a bridge. They rely on suspended large masses tuned to the primary frequency of the structure. Very recently "active" tuned mass dampers have been used in buildings. The mass is activated by motors controlled by a computer that is

programmed to recognize structural movements induced by wind or earthquake. They probably have little application to the Golden Gate Bridge.

7.2.3 Structural Fuses And Lock-Up Devices

Structural fuses and lock-up devices act in opposite ways. A structural fuse connects separate structural units under ordinary loads. In the event of an earthquake or high wind, the fuse breaks and allows the single structure to act as two independent systems.

A lock-up device allows two separate structural units to operate as separate units under ordinary loads and temperature. In the event of an earthquake, the device locks-up and allows the entire structure to resist the seismic forces as a unit. The original Carquinez Bridge and the new Dumbarton Bridge both have this kind of device installed. A secondary requirement is that the device must have low maintenance requirements and must function when it is needed. The lock-up device can generate large local forces in the structure.

Structural fuses may not have any application to the Golden Gate Bridge but the lock-up devices may warrant further consideration.

7.2.4 Changes In Articulation And Composite Action

The two structural response modifications that may have the potential for practical application for superstructural modification leading to possible cost savings are changes in the articulation of the structure and joining components in composite action.

One possible application of changed articulation is making the approach span superstructures one continuous structure. Large stresses due to temperature movements could be a secondary effect.

Another possible application involves moving the fixed pin of the side span of the suspension bridge from its present location at the main towers to the opposite end to the adjacent concrete pylons. This change in articulation may reduce stress in the main towers but the secondary effect will be the generation of large forces in the pylons. The pylons need retrofitting anyway and with a little additional reinforcement, the pylons may be able to take the increase in loading.

There are at least two areas of the bridge where composite action may have practical advantages as a possible means of seismic upgrading and should be investigated. The center portion of the side span stiffening trusses of the suspension bridge have high chord stresses. A possible solution is to make the pedestrian sidewalk work compositely with the upper chords of the stiffening

trusses. The concrete walkway would have to be replaced with a steel plate and the steel plate tied in at each panel point to make an effective composite action system. The steel plates would require a surfacing to make them waterproof and skid resistant.

The second possible composite action solution involves adding a new section of the proposed transit deck on the plane of the lower chords. The transit deck as proposed is an orthotropic steel deck that can be made to act compositely with the lower chords of the stiffening truss. This may cost a little more but would be a start on adding the transit system to the bridge.

Another possible area of composite action is in the north and south approach truss spans. There the concrete sidewalk can be replaced with steel plates, and connected to the chords in the same manner as for the suspended side spans.

7.3 Summary

Many methods presently exist and others can be devised to seismically upgrade the Golden Gate Bridge. Each needs to be carefully investigated, and feasibility and costs determined. This step is necessary before construction plans can be produced to do the actual retrofitting work.

7.4 Cost Of Seismic Upgrading

The primary emphasis of this seismic evaluation was to determine the seismic vulnerability of the Golden Gate Bridge. Estimating the cost of upgrading the vulnerable areas of the bridge is difficult as no construction details have been worked out. Without these construction details, cost estimates are only best judgments.

Our best judgment at this time places the cost of seismically upgrading the Golden Gate Bridge at \$100 million plus or minus \$25 million.

8. CONCLUSIONS

8.1 VULNERABILITY

Vulnerable components of the bridge were identified by detailed review and evaluation of the computer analyses. The degree of vulnerability varied greatly between the various structural subsystems of the bridge.

The bridge approaches, consisting of the Fort Point arch and the steel girder and truss viaducts supported on braced frames, are among the most vulnerable parts of the bridge. In a Richter magnitude 8 or greater earthquake centered near the bridge, high levels of damage can be expected in these structures. After such an earthquake, it is expected that these viaducts would no longer be able to carry traffic. There is substantial risk of collapse or impending collapse after such an event. The pylons and anchorage housings, of reinforced concrete construction, would experience damage similar to that of the approach viaducts. The probable high level of damage to these structures would also affect the performance of the main bridge. In a Richter magnitude 7 to 7.5 earthquake centered near the bridge, damage would be somewhat less severe, but significant repairs would still be required.

The 6450-foot long suspension bridge is also vulnerable to damage during a Richter magnitude 8 or greater earthquake centered near the bridge. Critical components include the reinforced concrete piers supporting the steel towers, the bases of the towers above where they are anchored to the piers, the connections between the towers and the stiffening trusses, the saddles connecting the main cables to the tower tops, and the chords of the side-span stiffening trusses. The cable tie-downs in the pylons are also quite vulnerable, and their damage could lead to further secondary damage in the rest of the bridge. Most other structural components of the main bridge are in less danger, but the roadway expansion joints could be subject to secondary damage.

With the bridge in its current state, a Richter magnitude 8 or greater earthquake centered near the bridge, similar to the 1906 event, would likely cause some damage over the entire length of the bridge, with the risk of collapse of parts of the approaches. In the event of a collapse, this vital transportation link would be out of service for an extended period.

A Richter magnitude 7 to 7.5 earthquake on the Hayward or San Andreas fault, similar to the Loma Prieta event but centered near the bridge, would cause somewhat less damage, but could also result in temporary closure of the bridge during repairs.

8.2 RECOMMENDATIONS

While the Golden Gate Bridge has performed well in previous earthquakes, it is vulnerable to damage in a Richter magnitude 7 or greater earthquake with epicenter close to the bridge, and it could be closed for repair for some time after such an earthquake. The Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake has recommended that transportation structures of regional importance, such as the Golden Gate Bridge, should be retrofitted so that such closure is not required. This Board also recommended instrumenting such structures to record ground motion and structural response. Both these measures should be undertaken.

Structural retrofitting of the bridge prior to a strong earthquake could minimize the potential for damage and could eliminate the need for long-term closure for repairs after the earthquake. Retrofitting of all the vulnerable components would be required to minimize the risk such an earthquake poses. The retrofit measures can be built without closure of the bridge to traffic, using methods similar to those used during the recent deck replacement project.

The steel approach viaducts need to be made two to six times stronger with respect to seismic resistance to provide confidence that the bridge would remain open after a Richter magnitude 7 or greater earthquake centered near the bridge. The most critical components for retrofit are the foundations, supporting frames, structural bearings supporting the trusses, and the expansion joint restrainers. The trusses supporting the roadway also require strengthening, but are somewhat less critical.

The reinforced concrete anchorage housings and pylons need a similar level of strengthening. Such strengthening could be similar to the seismic reinforcement of older buildings in San Francisco, consisting of strengthening the foundations, building new internal framing, and bracing the exterior walls.

The main suspension bridge requires strengthening of its reinforced concrete piers and lower tower sections, restraint of the tower-pier connections, strengthening of the side span stiffening trusses and the connections between these trusses and the towers, strengthening of the connections of the cable saddles to the tops of the towers, and extending the tie-downs in the pylons to a more secure anchorage in the underlying rock. Additional structural modifications would alleviate secondary damage due to large movements of the bridge during the earthquake.

An order-of-magnitude cost of about \$75- to \$100- million (about ten percent of the replacement cost of the bridge) is being estimated for the complete strengthening of the Golden Gate Bridge and its approach viaducts.

9. ACKNOWLEDGEMENTS

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T. Y. Lin International is the prime consultant to the District for the seismic evaluation. Principal-In-Charge Charles Seim assembled and directed the team, and provided liaison with the District. Project Manager Dr. Mark Ketchum performed or supervised technical analysis and engineering efforts. Project staff includes Ron Zimmerman, Al Heldermon, Kar Seah, Dr. Albert Tung, and J. T. Teng. Technical advice was provided by Professor Alexander Scordelis and Professor Hassan Astaneh, both of the University of California at Berkeley. Special thanks are due to Professor Emeritus Frank Baron, of the University of California at Berkeley, who provided insightful remarks as well as a rich history of previous work on the Golden Gate Bridge.

Imbsen & Associates, Inc. was subconsultant assisting in the seismic evaluation of the main suspension bridge. Dr. Roy Imbsen and Dr. W. David Liu are responsible for IAI's contributions. Project staff includes F. S. Nobari and Dr. R. Hamidi.

Geospectra, Inc. was subconsultant for seismic risk and ground motion evaluation. Dr. J. P. Singh and Dr. Mansour Tabatabaie are responsible for Geospectra's contributions.

A board of review, consisting of recognized experts in the fields of seismology, structural dynamics, and earthquake engineering, provided an independent evaluation of the scope, methods, and findings of the evaluation. This board of review consists of Professor Bruce Bolt, Professor Joseph Penzien, and Professor Vitelmo Bertero, all of the University of California at Berkeley.

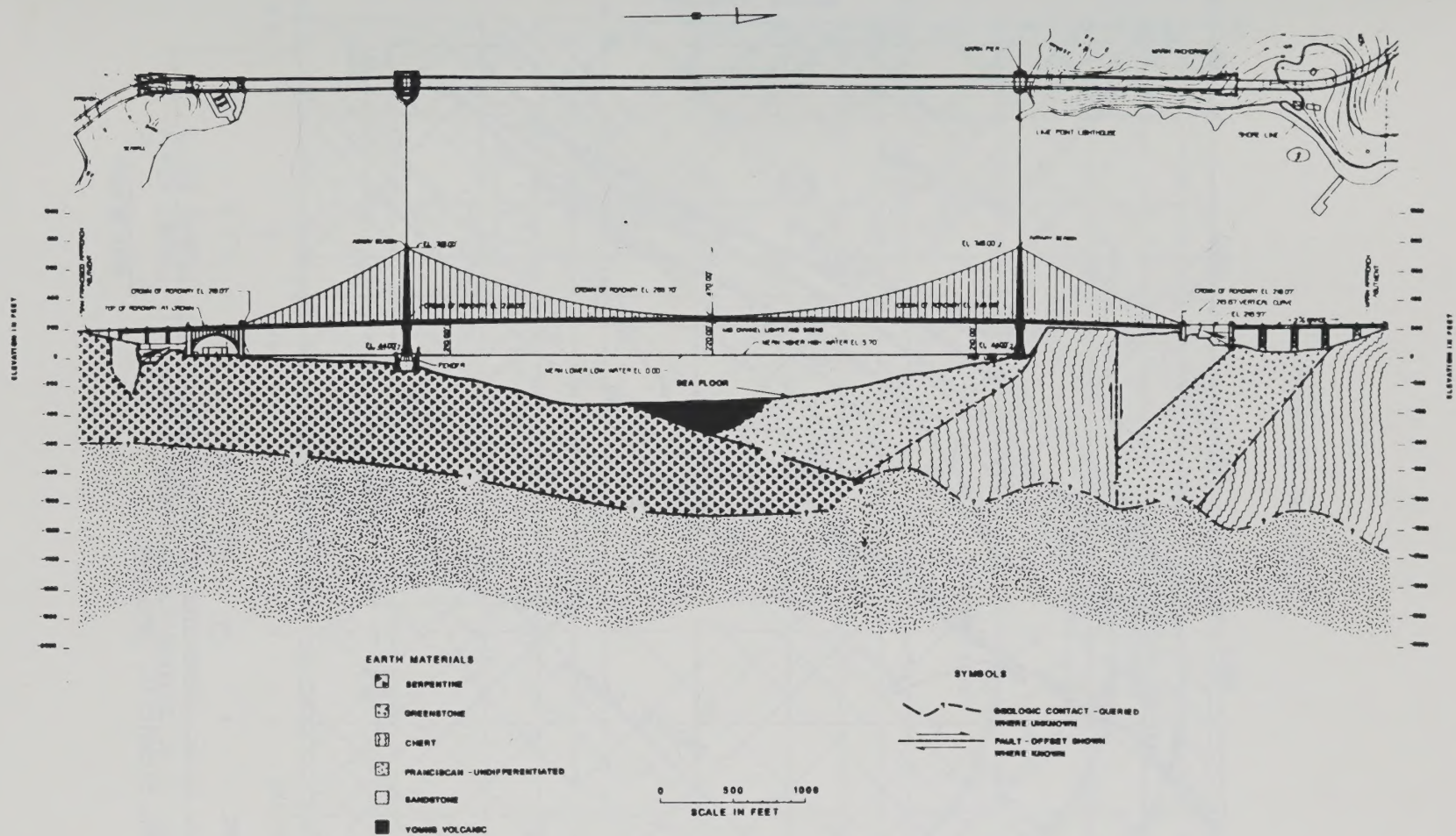
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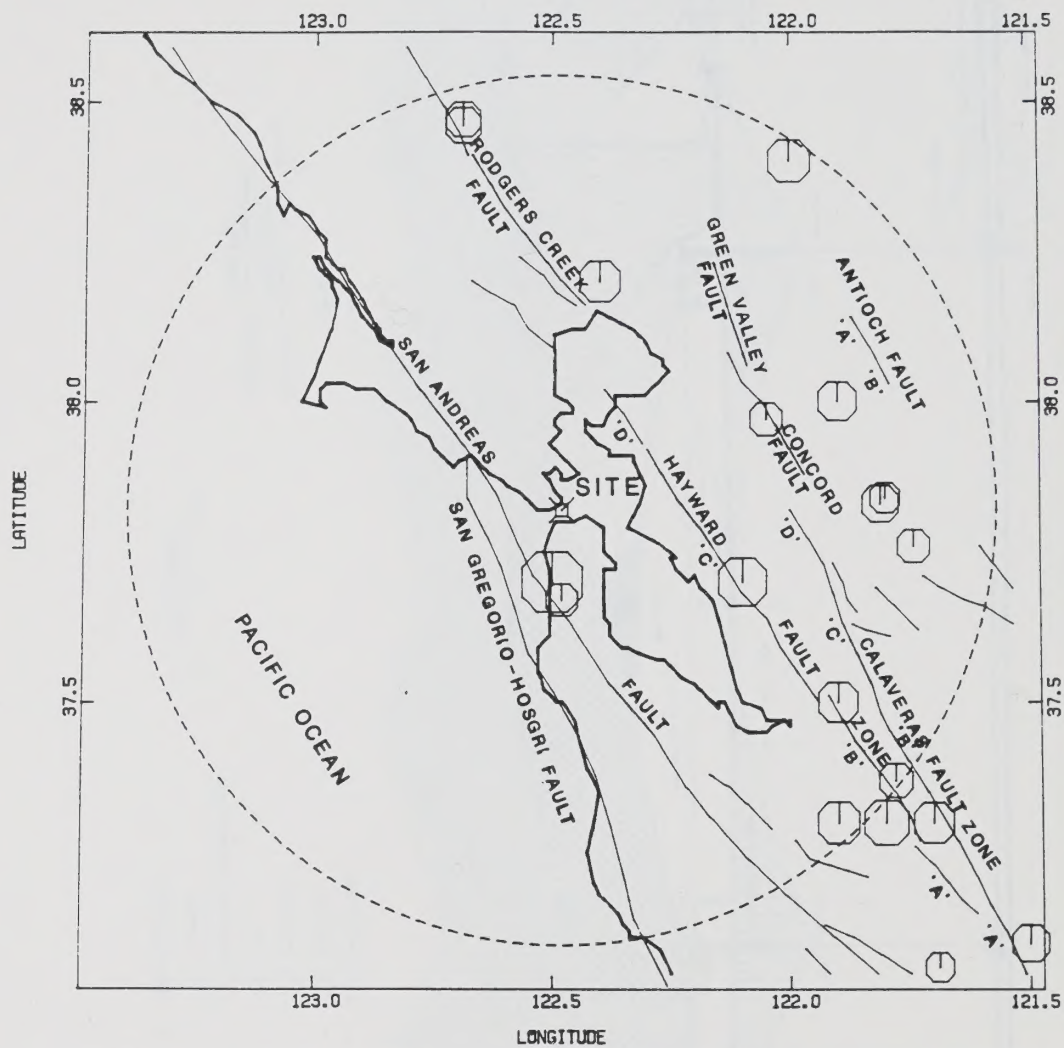
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APPENDIX A - GROUND MOTION RECORDS

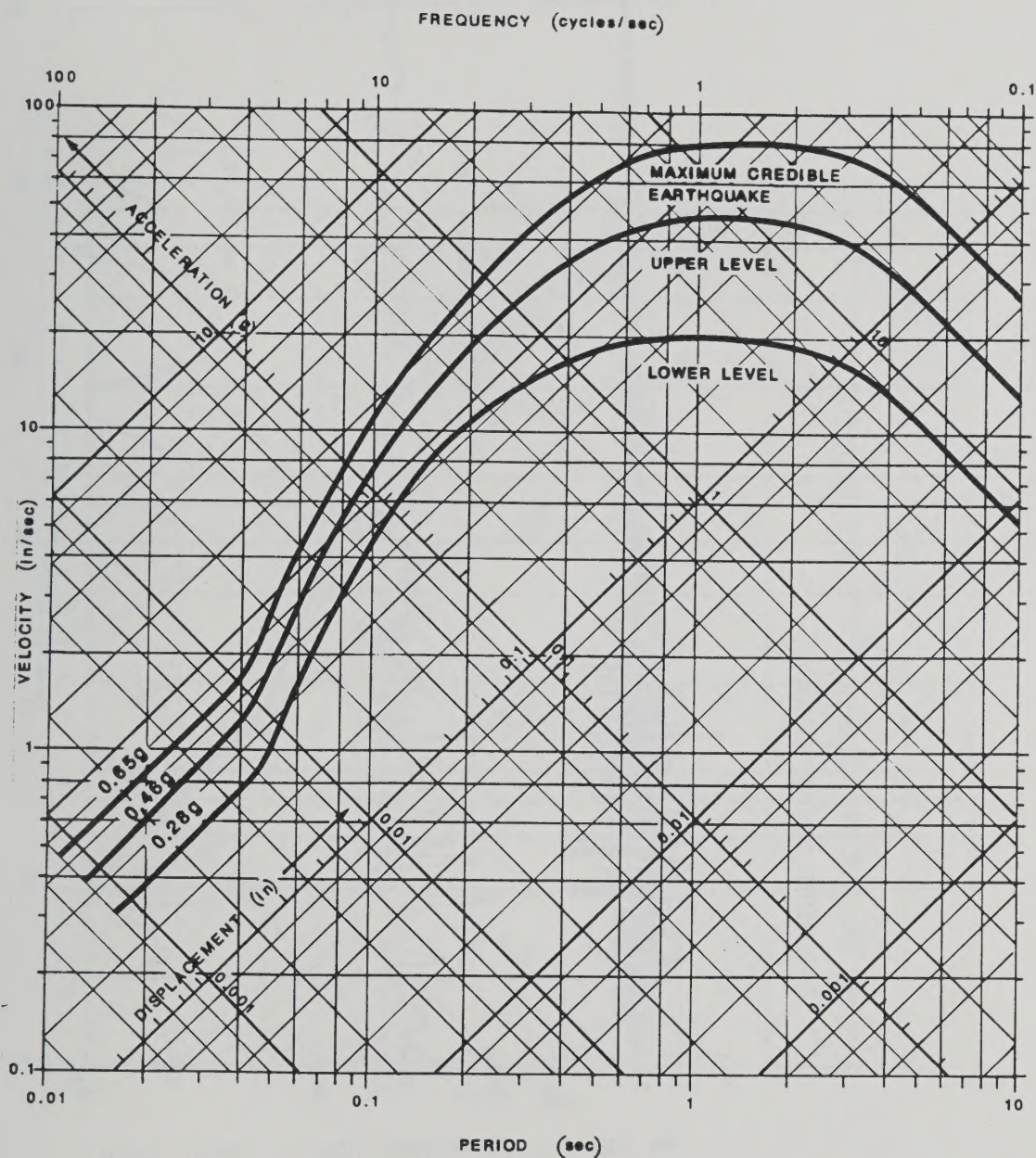




MAGNITUDE	:	4	5	6	7	8
SYMBOL	:					

REGIONAL SEISMICITY: 1850-1984

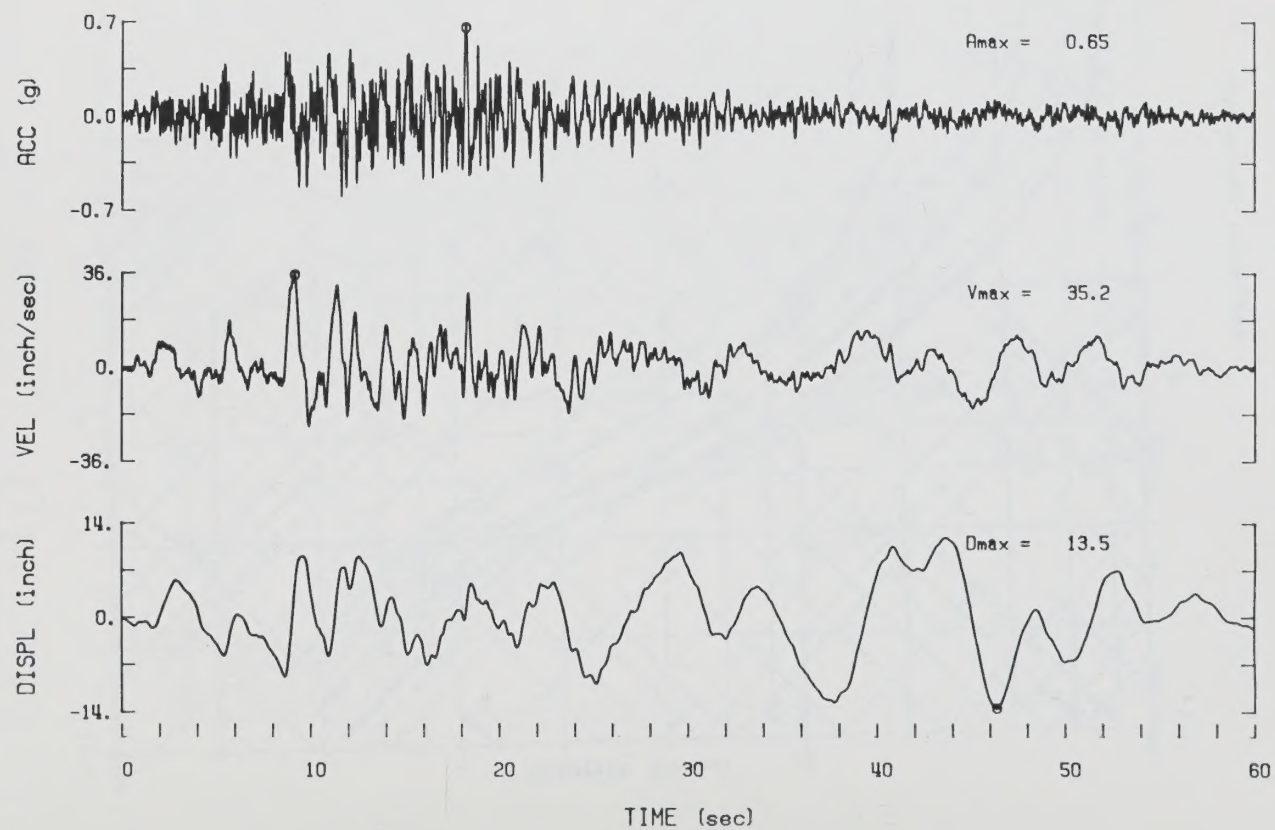
0. 20.
SCALE IN MILES

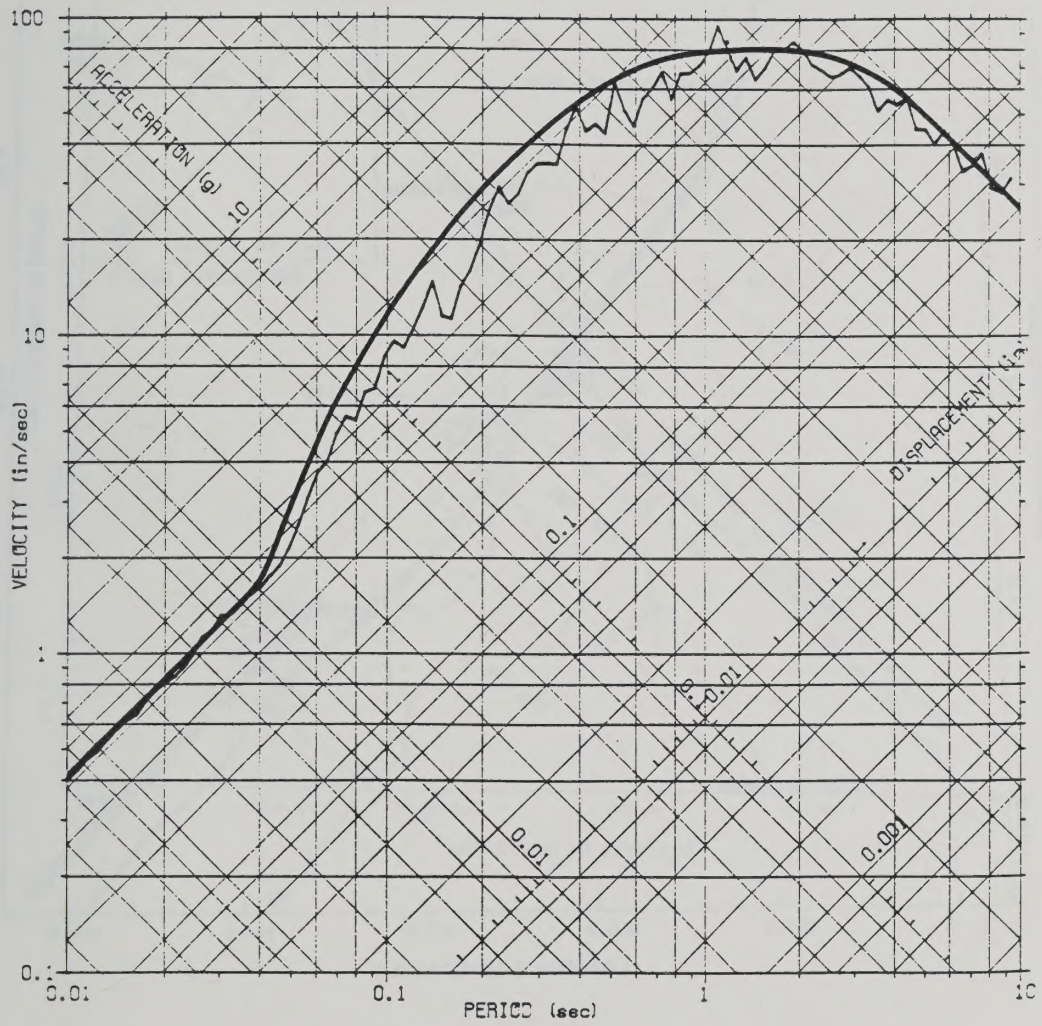


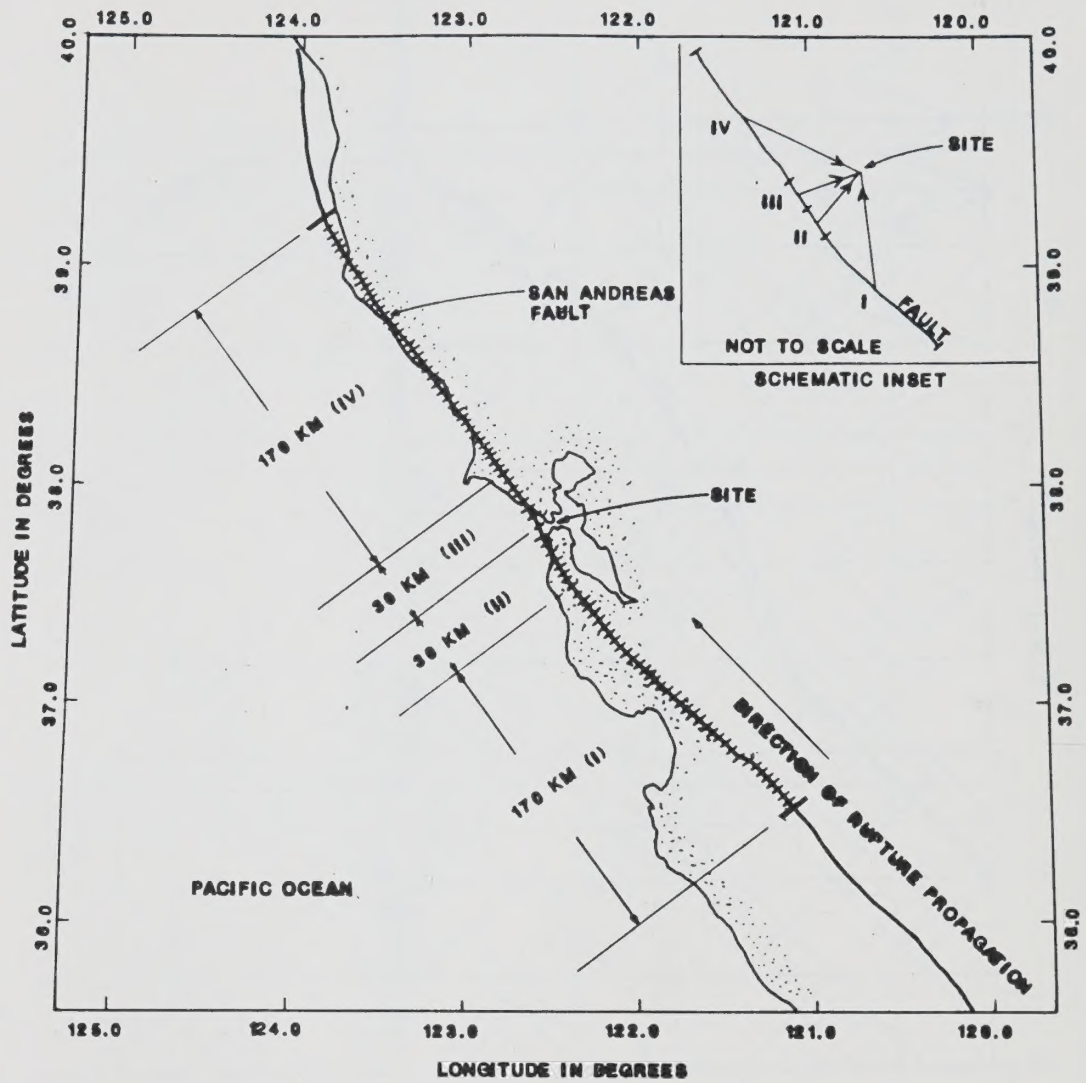
LOWER LEVEL: 50 PERCENT CHANCE OF EXCEEDENCE IN 50 YEARS
 UPPER LEVEL: 10 PERCENT CHANCE OF EXCEEDENCE IN 50 YEARS

GOLDEN GATE BRIDGE

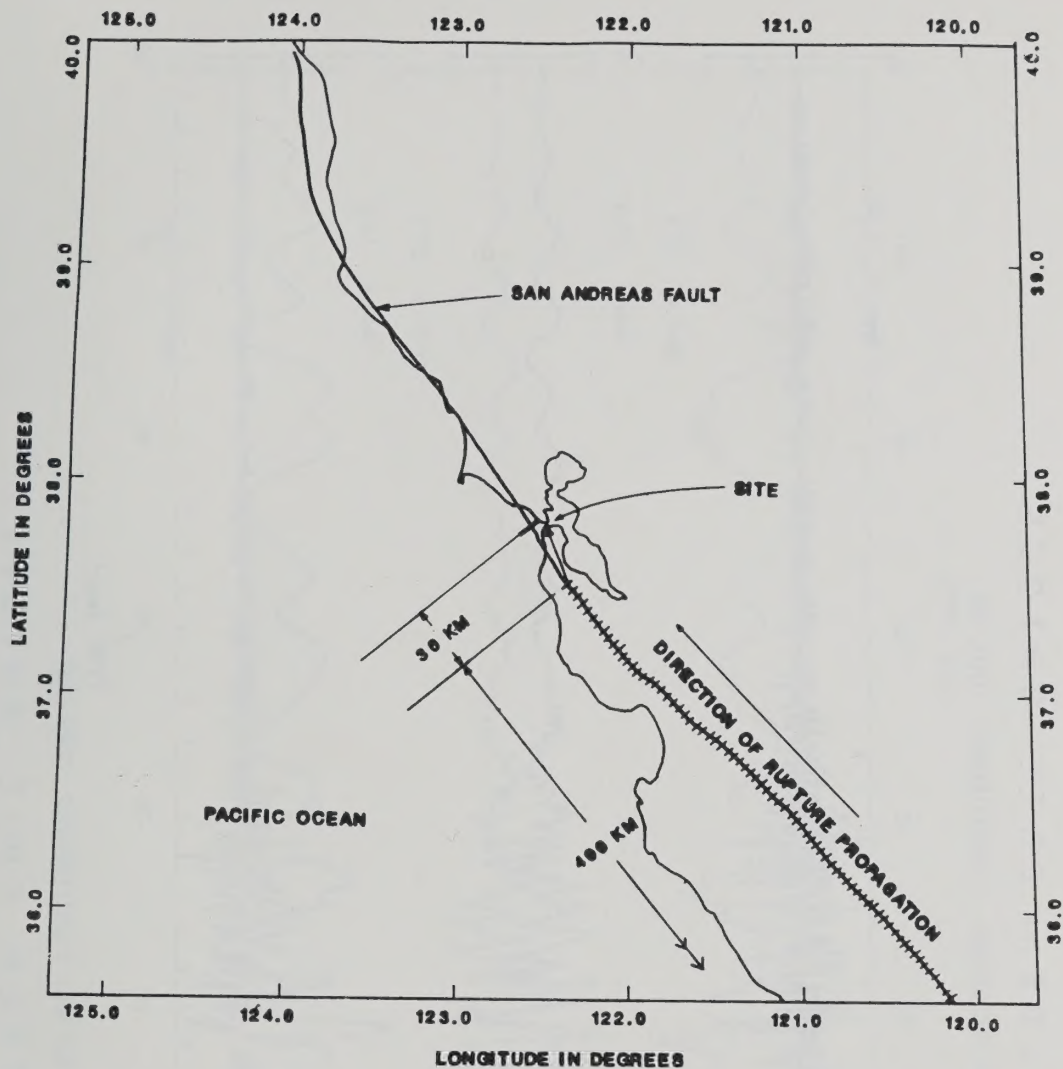
TRANSVERSE CONTROL MOTION





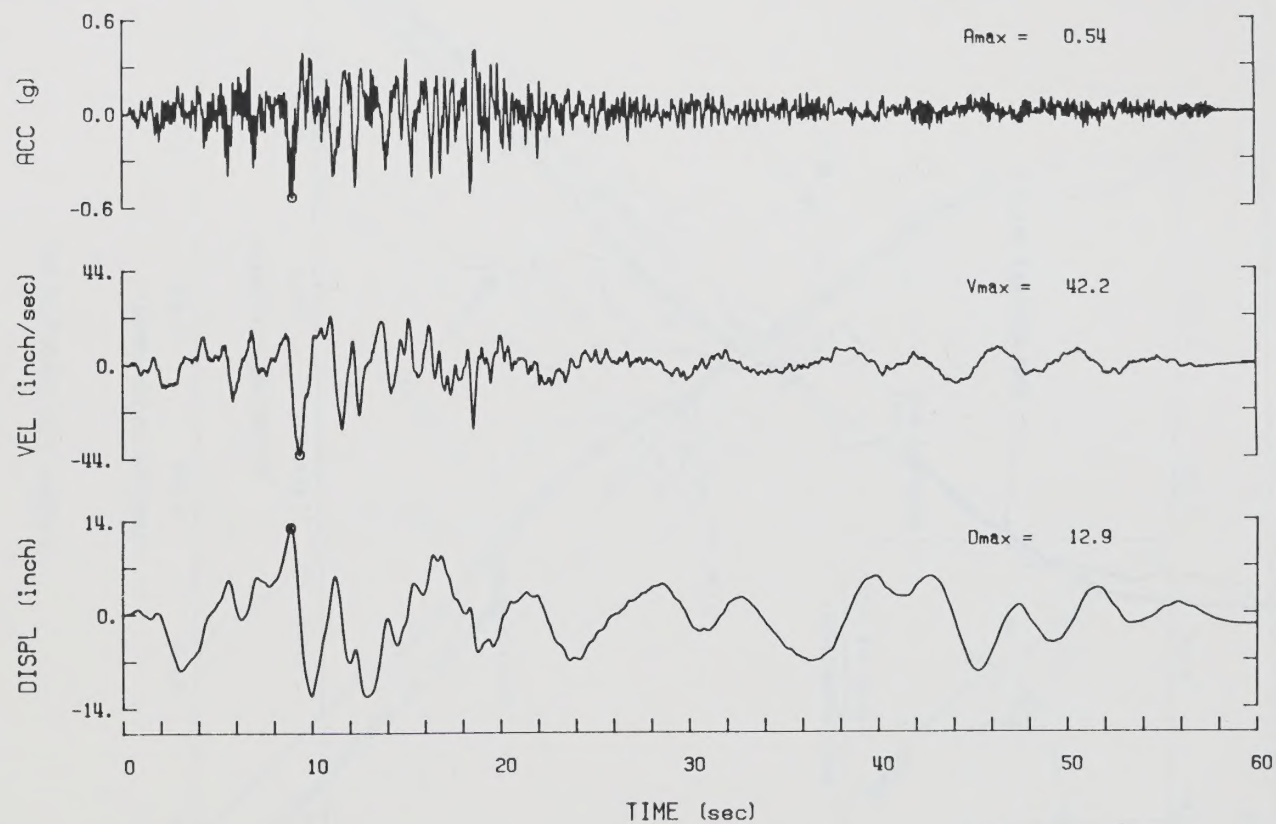


(A) RUPTURE SCENARIO 1



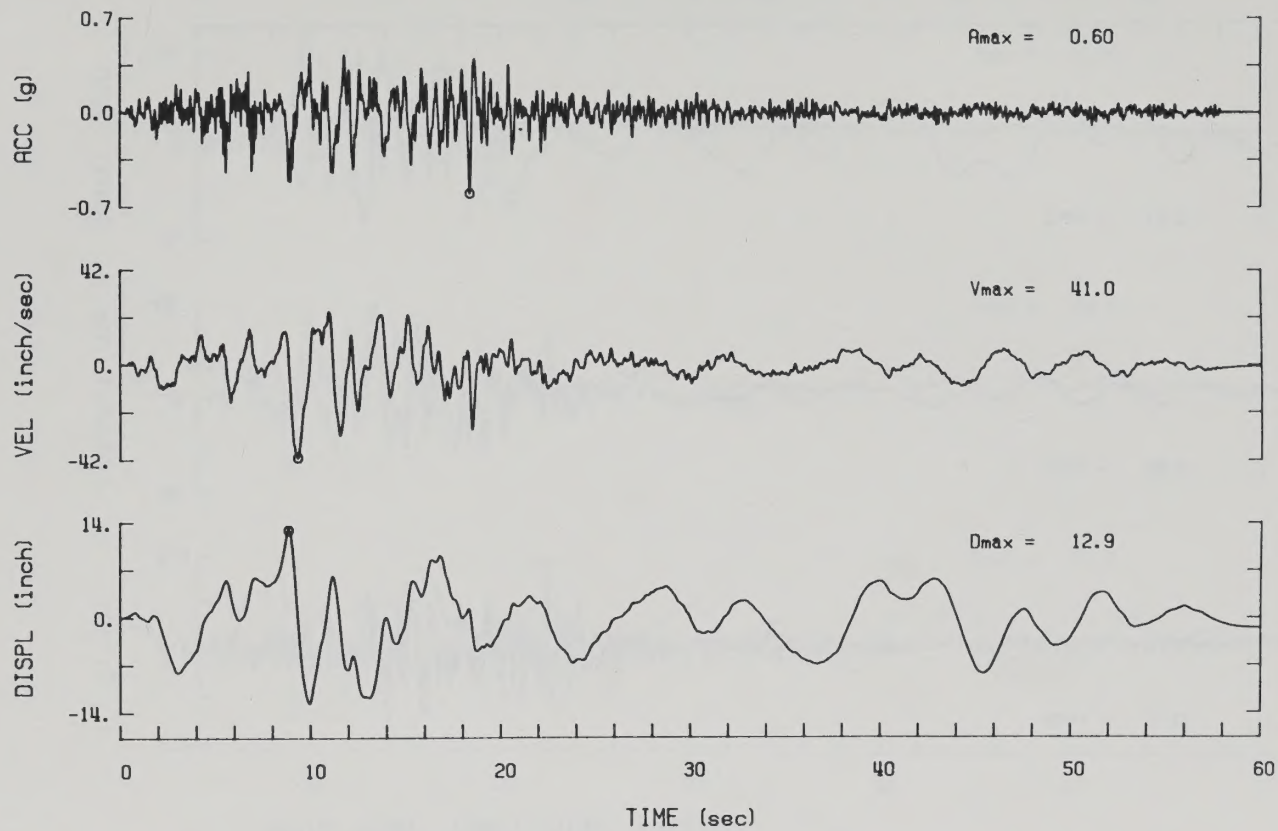
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MARIN ANCHORAGE - LONGITUDINAL (TH1L-4A)



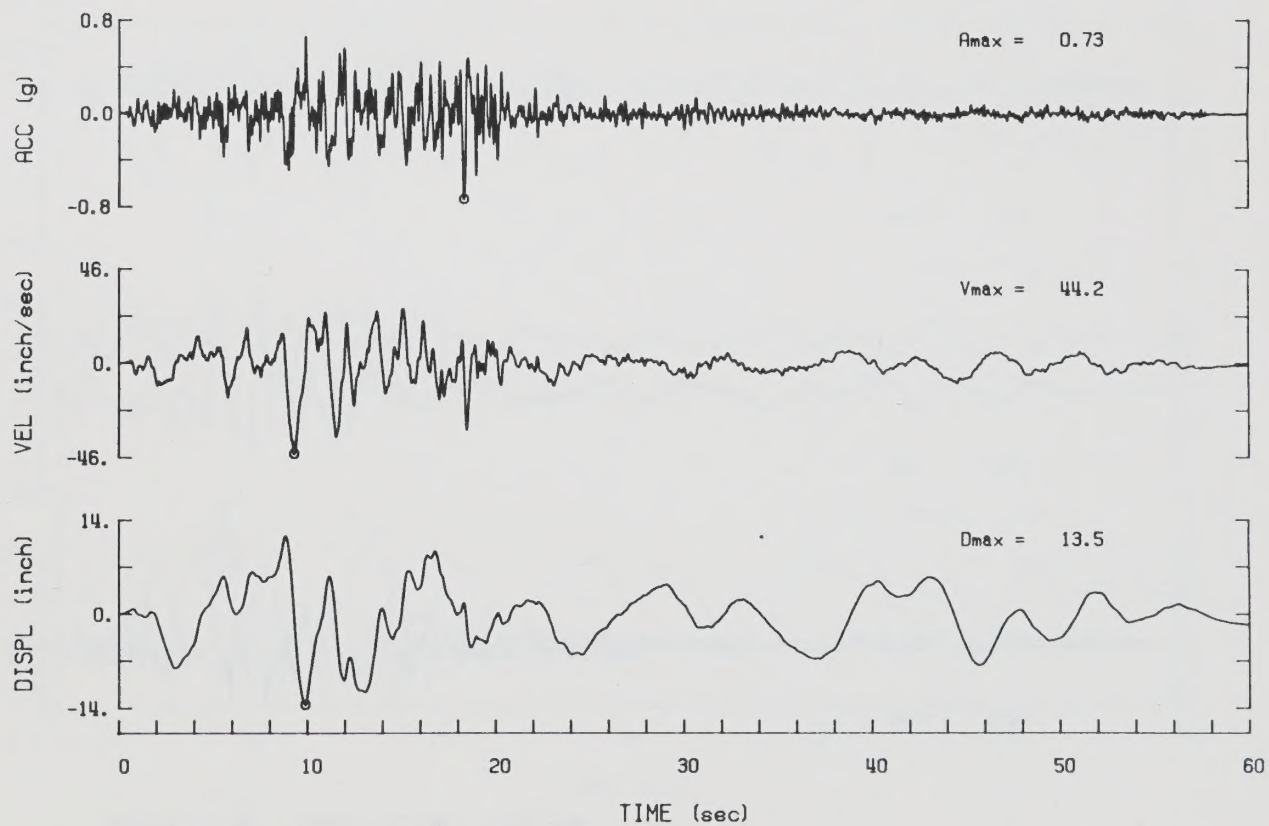
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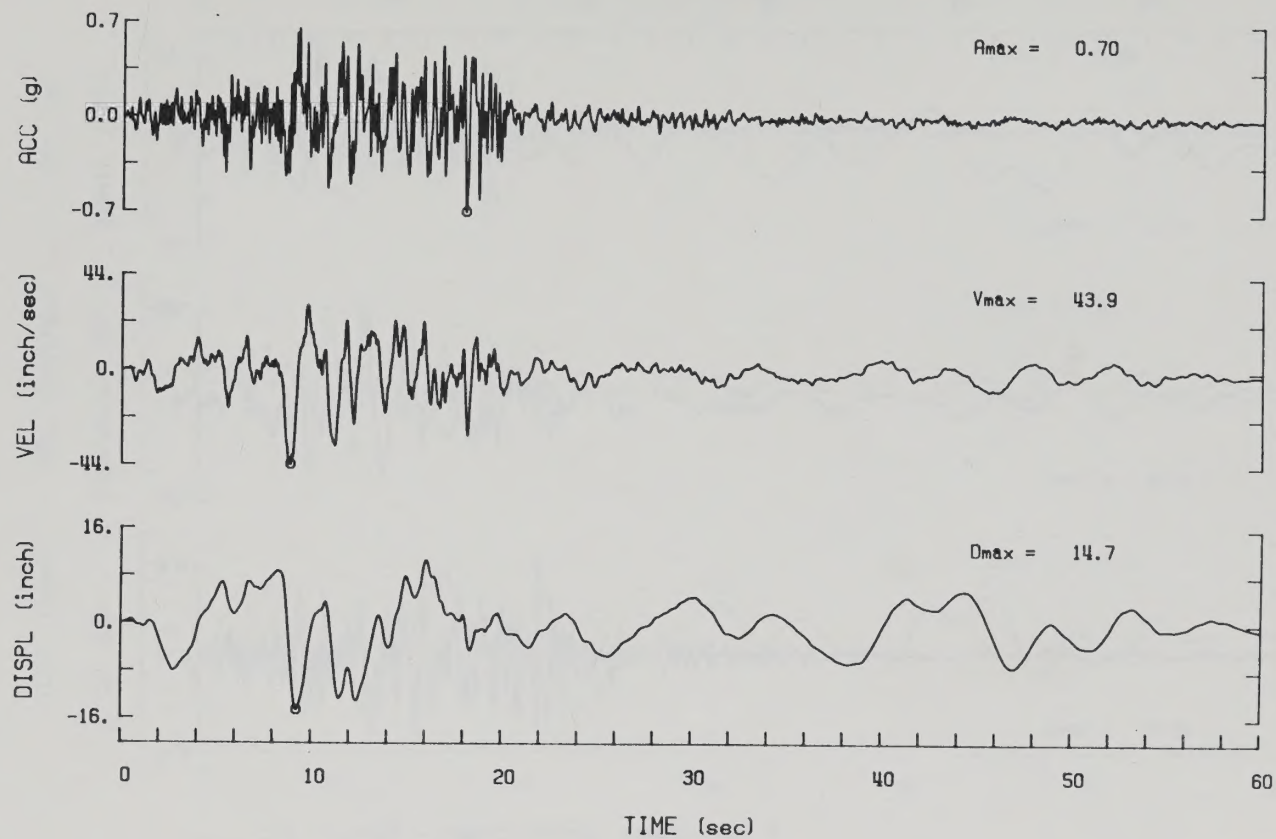
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MARIN TOWER - LONGITUDINAL (TH3L-4A)



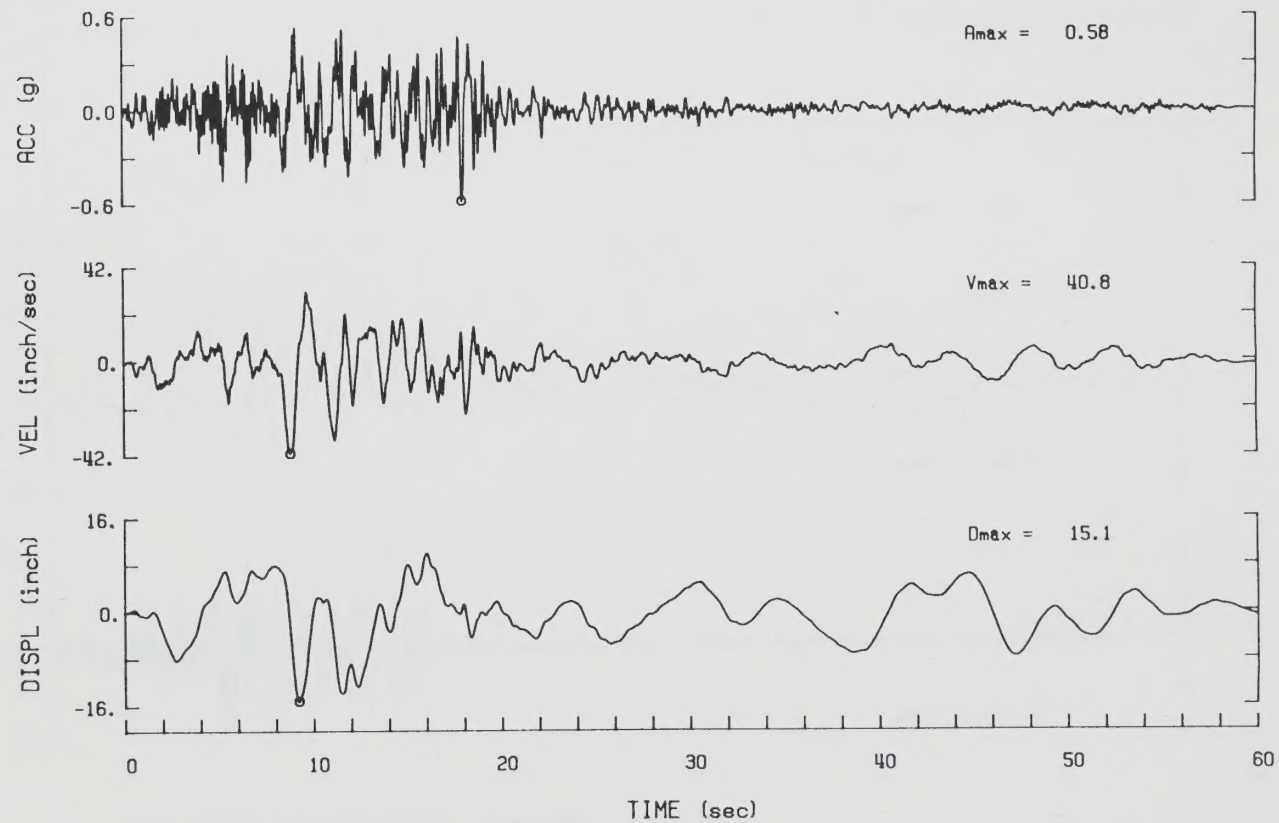
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S.F. TOWER - LONGITUDINAL (TH4L-4A)



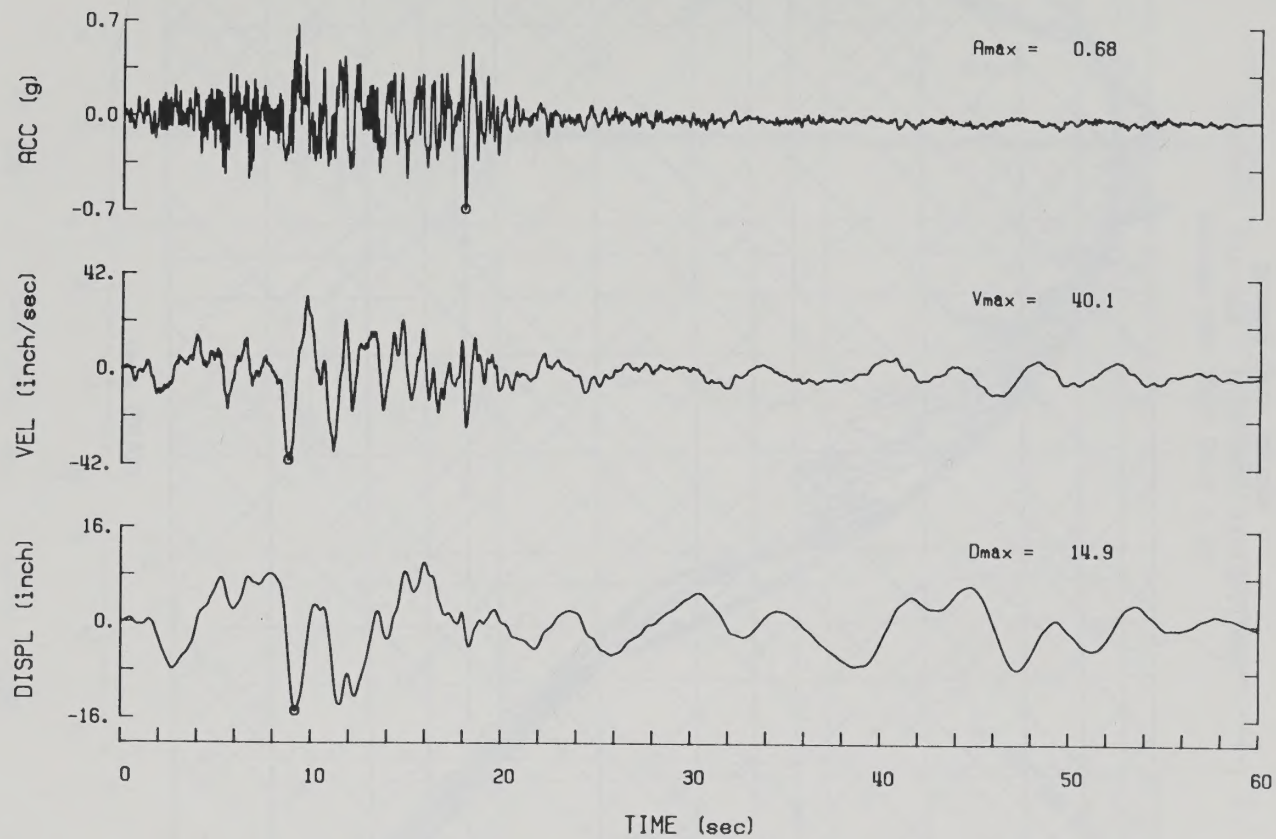
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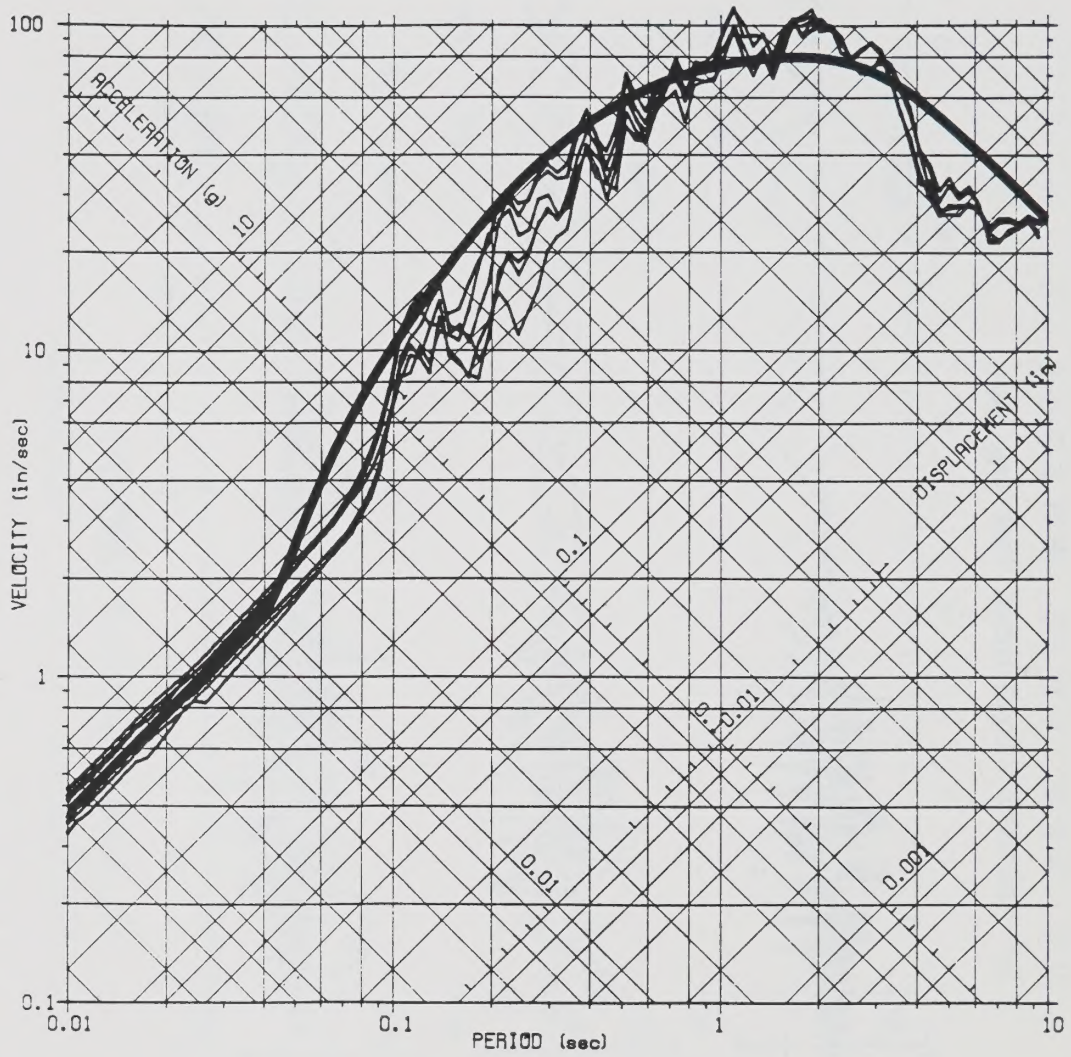


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S.F. ANCHORAGE - LONGITUDINAL (TH6L-4A)

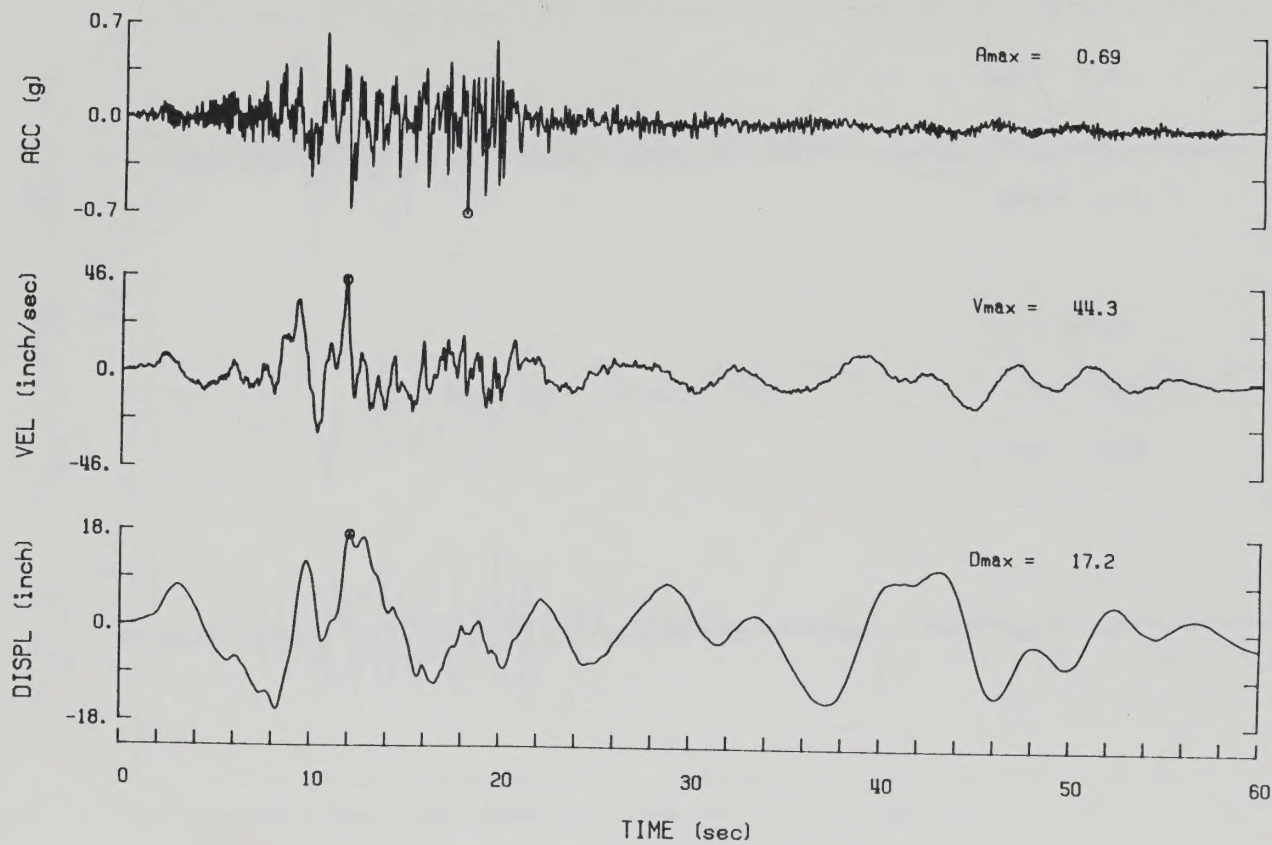


GOLDEN GATE BRIDGE
LONGITUDINAL COMPONENT - ALL SIX SUPPORTS
 $V_s = 4000$ FEET PER SECOND



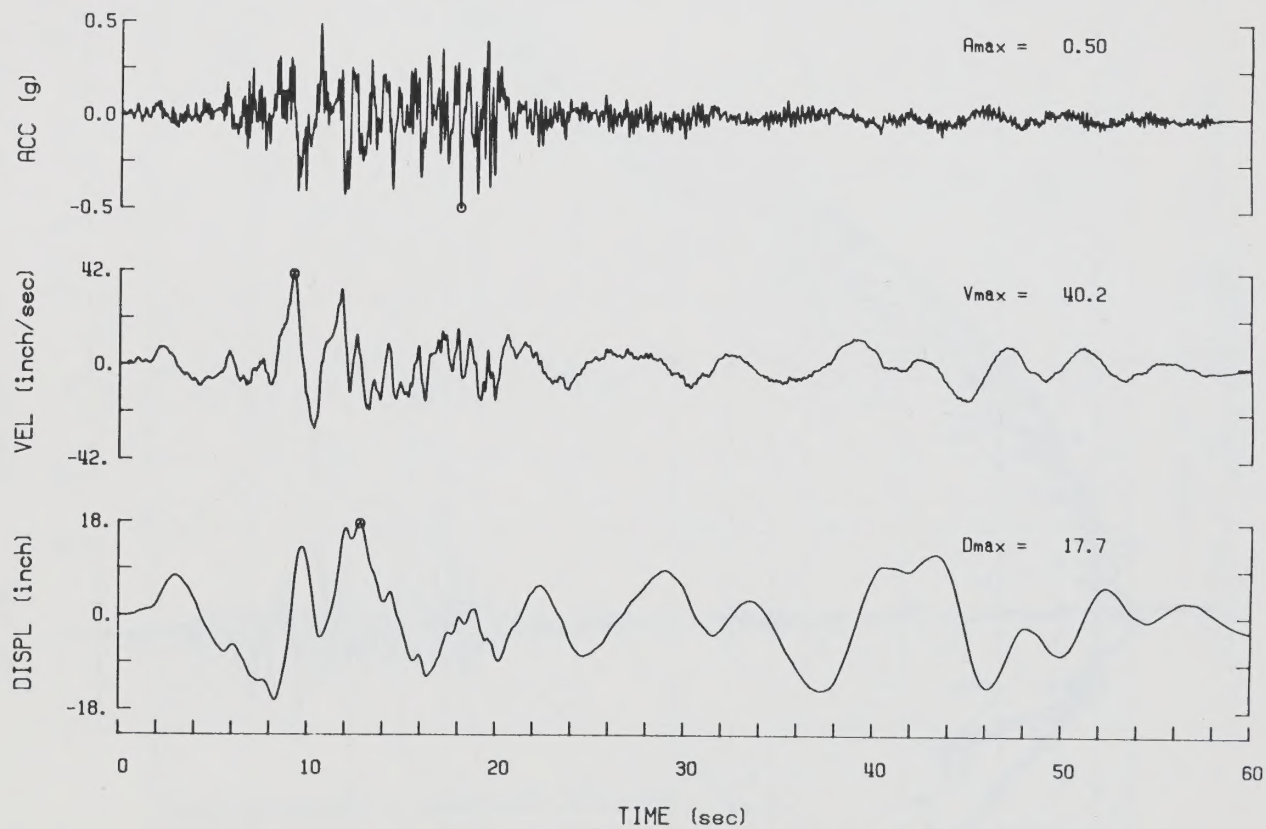
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MARIN ANCHORAGE - TRANSVERSE (TH1T-4A)



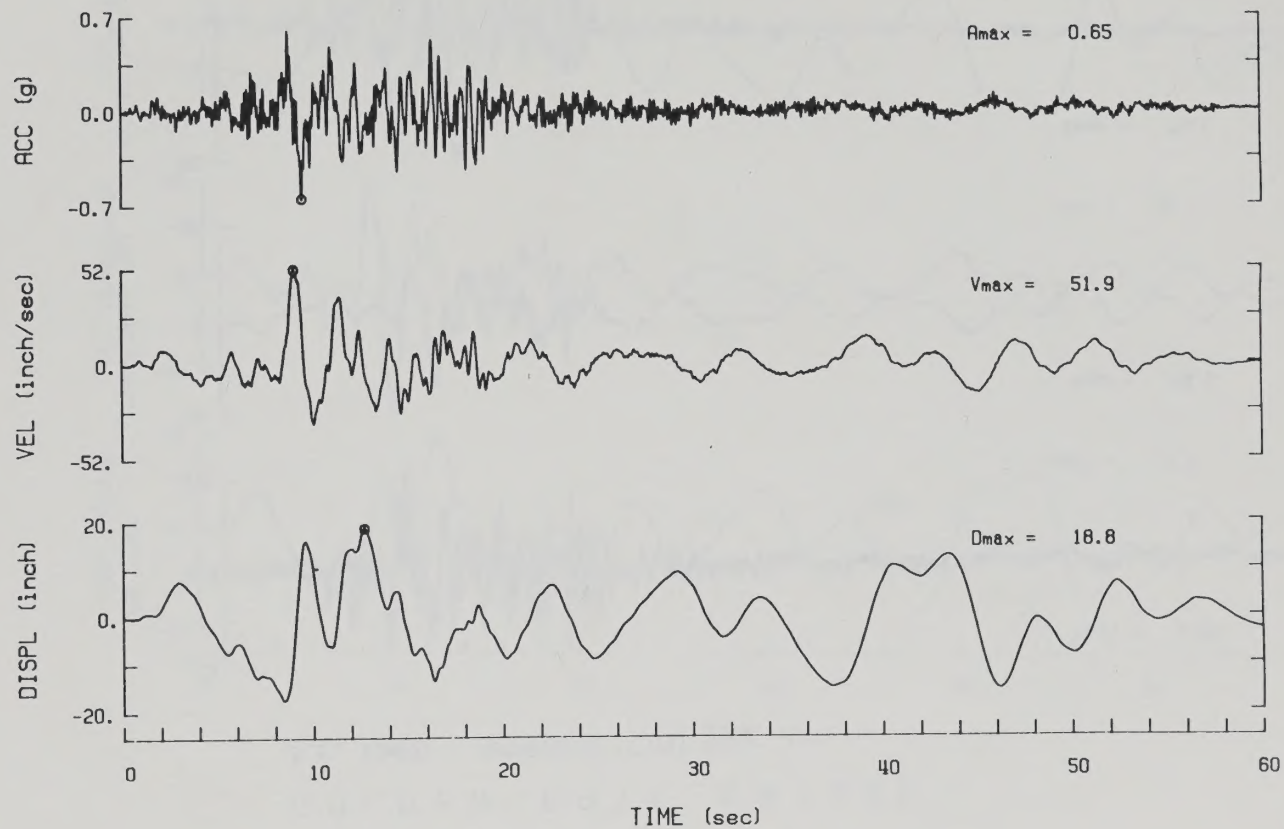
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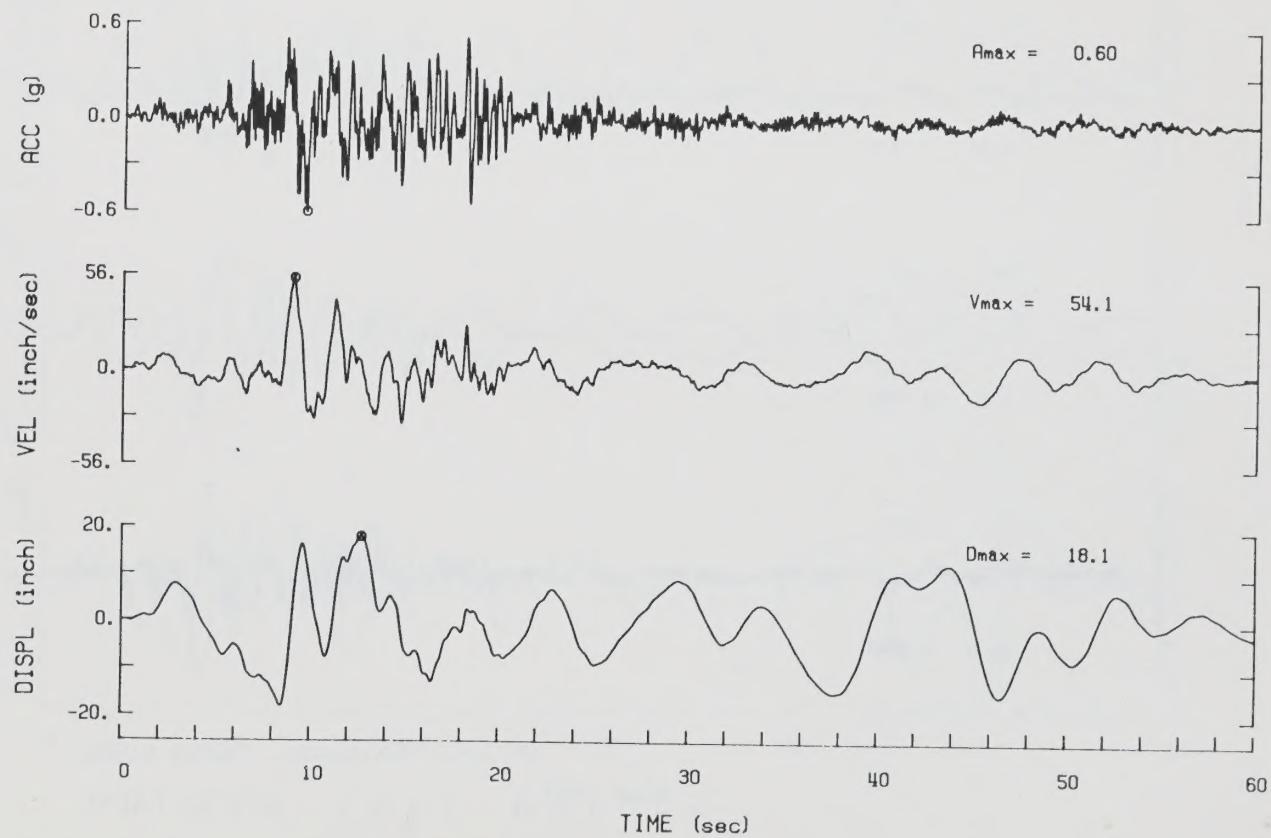
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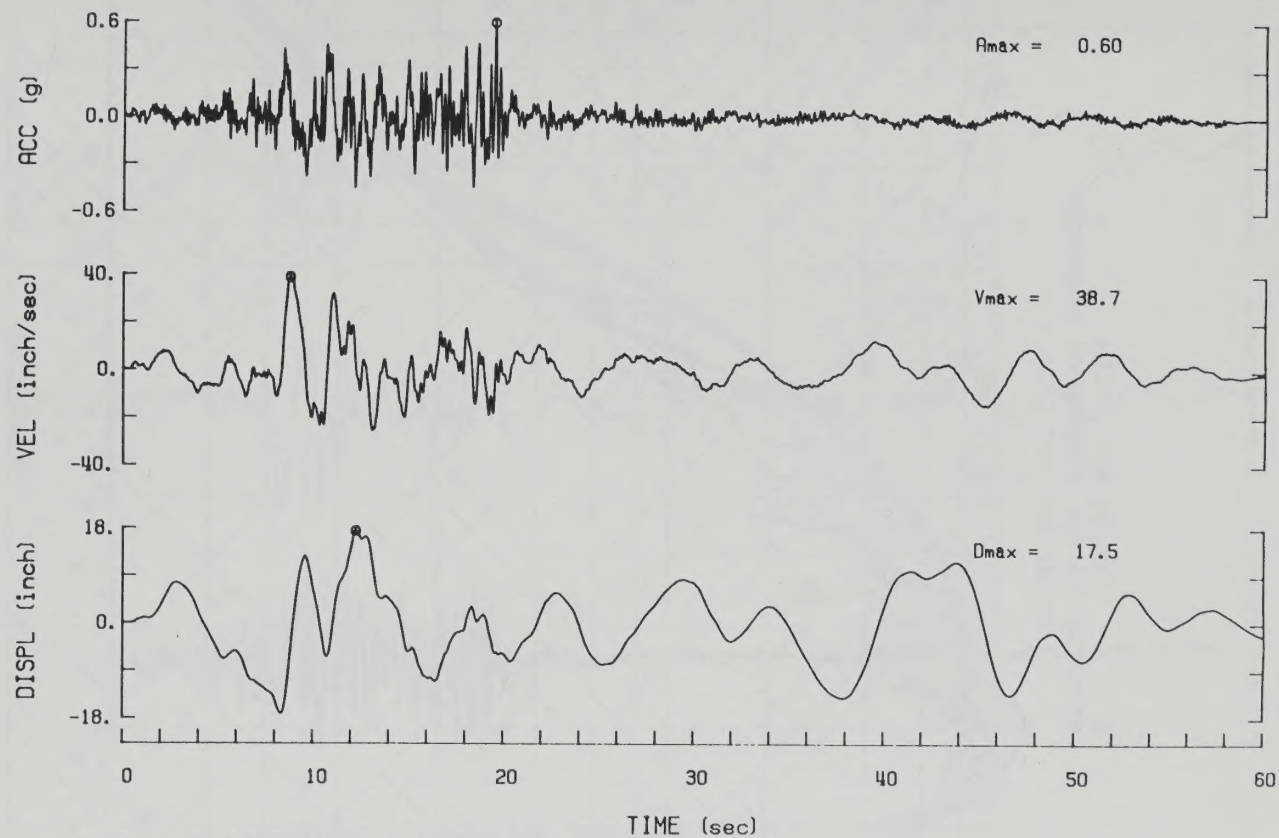
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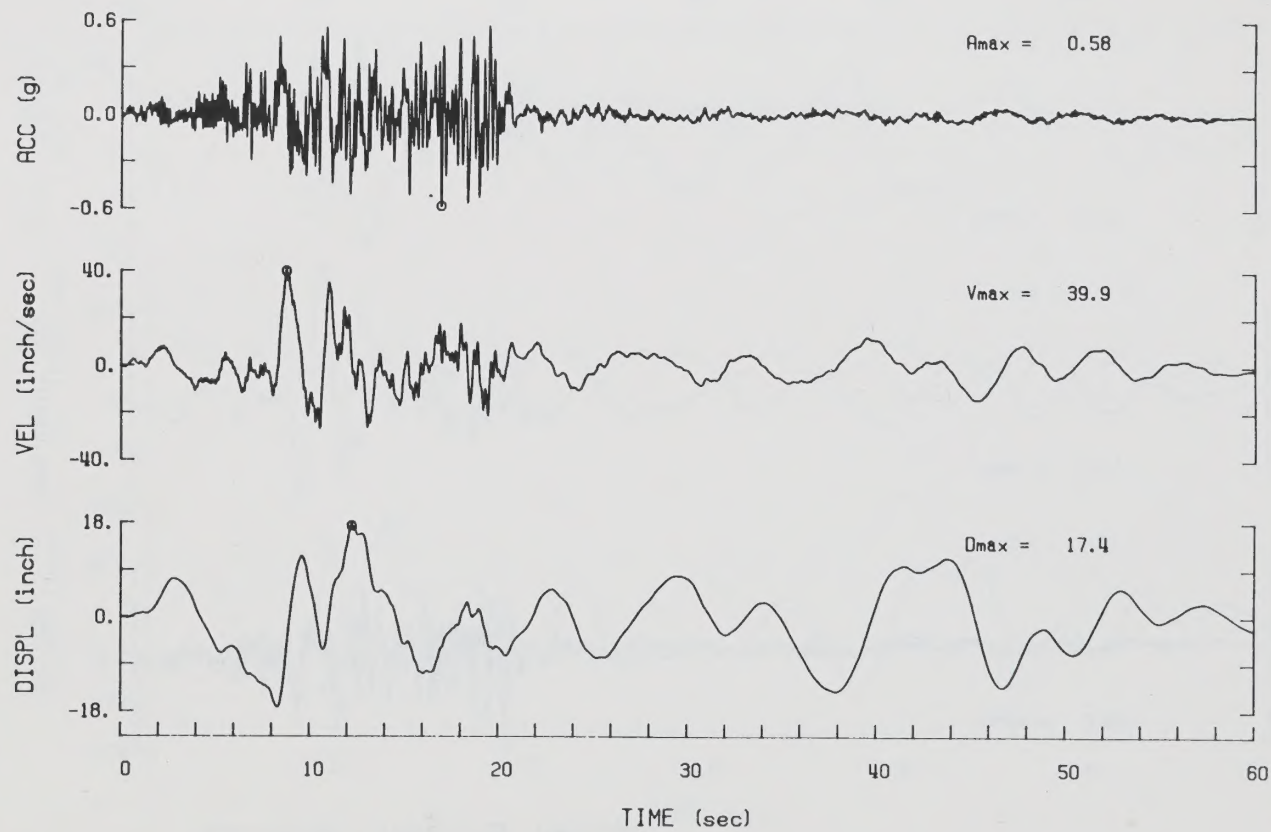
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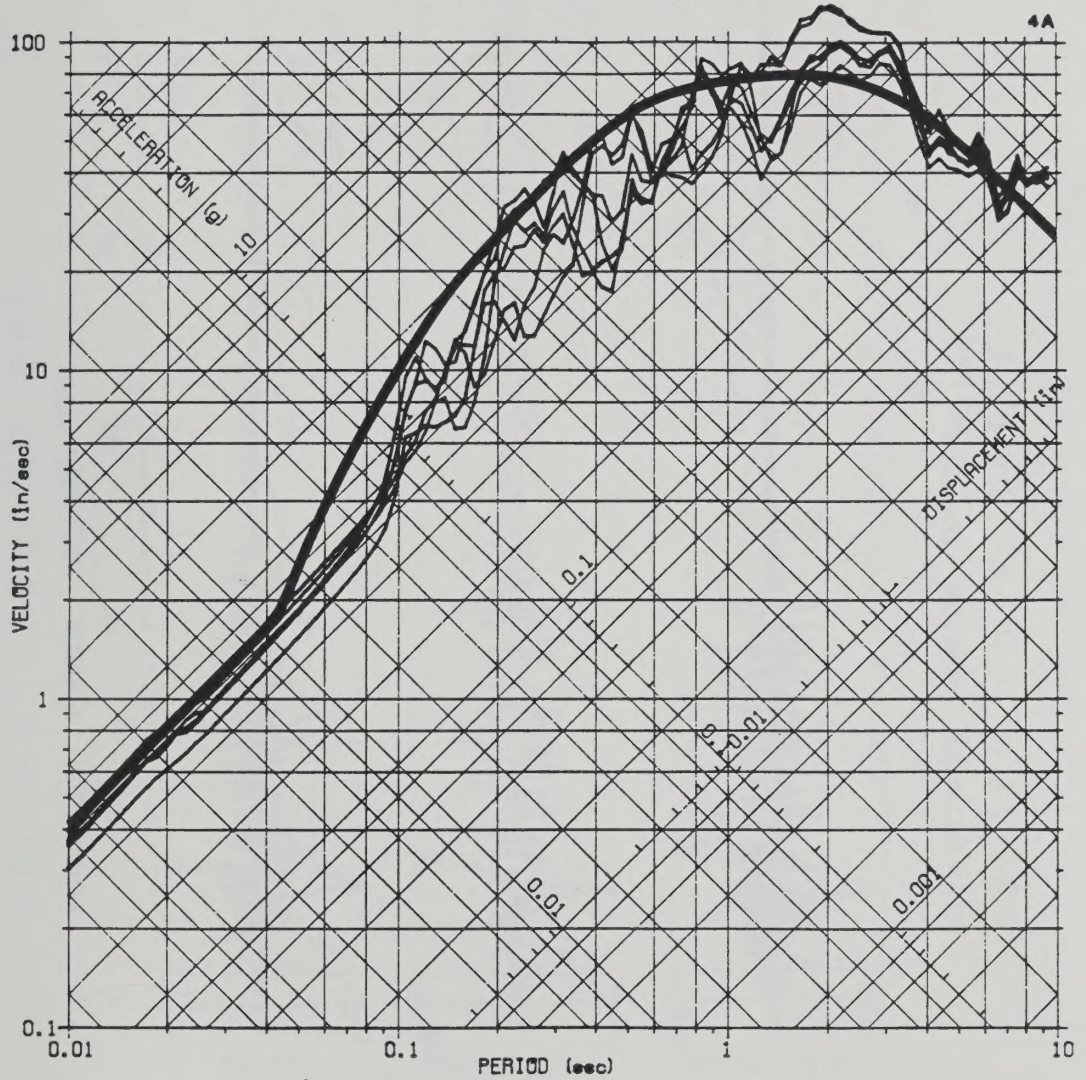


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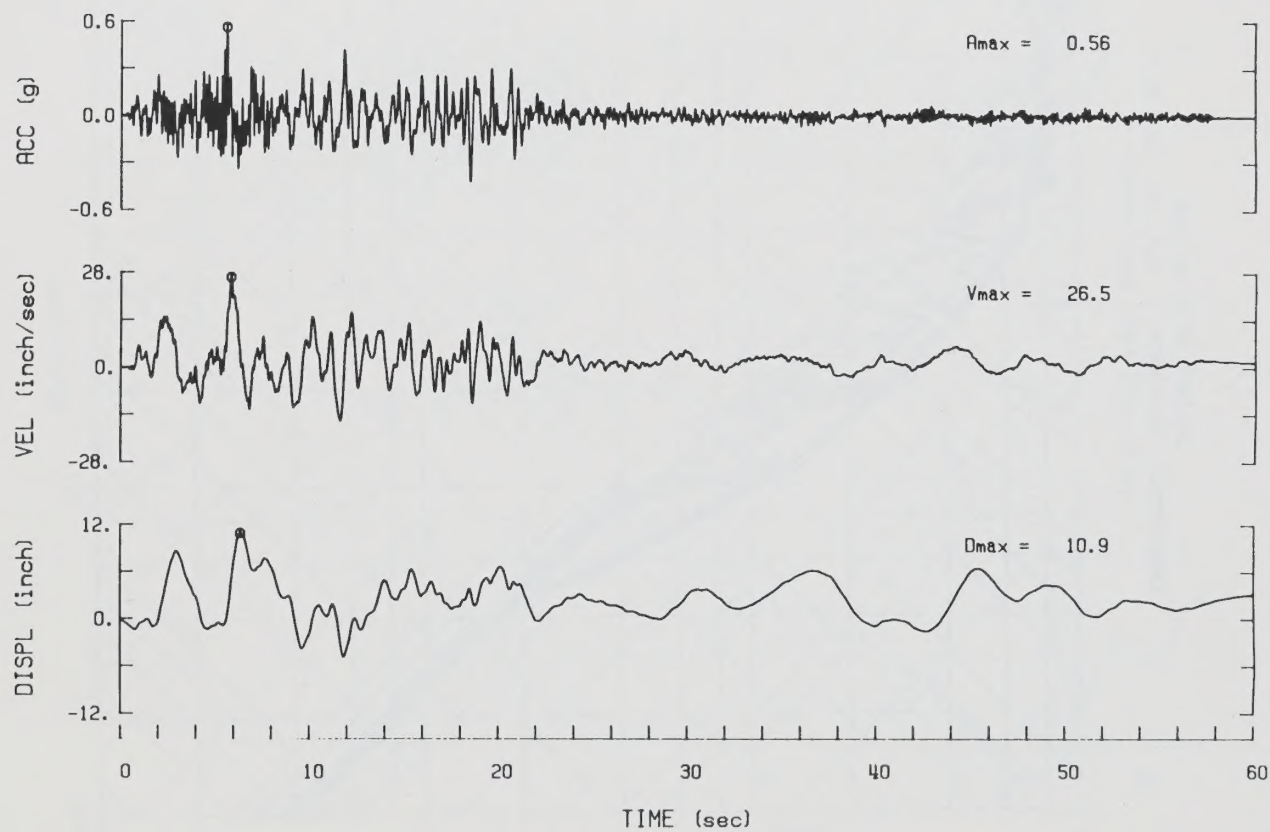


GOLDEN GATE BRIDGE
 TRANSVERSE COMPONENT - ALL SIX SUPPORTS
 $V_s = 4000$ FEET PER SECOND



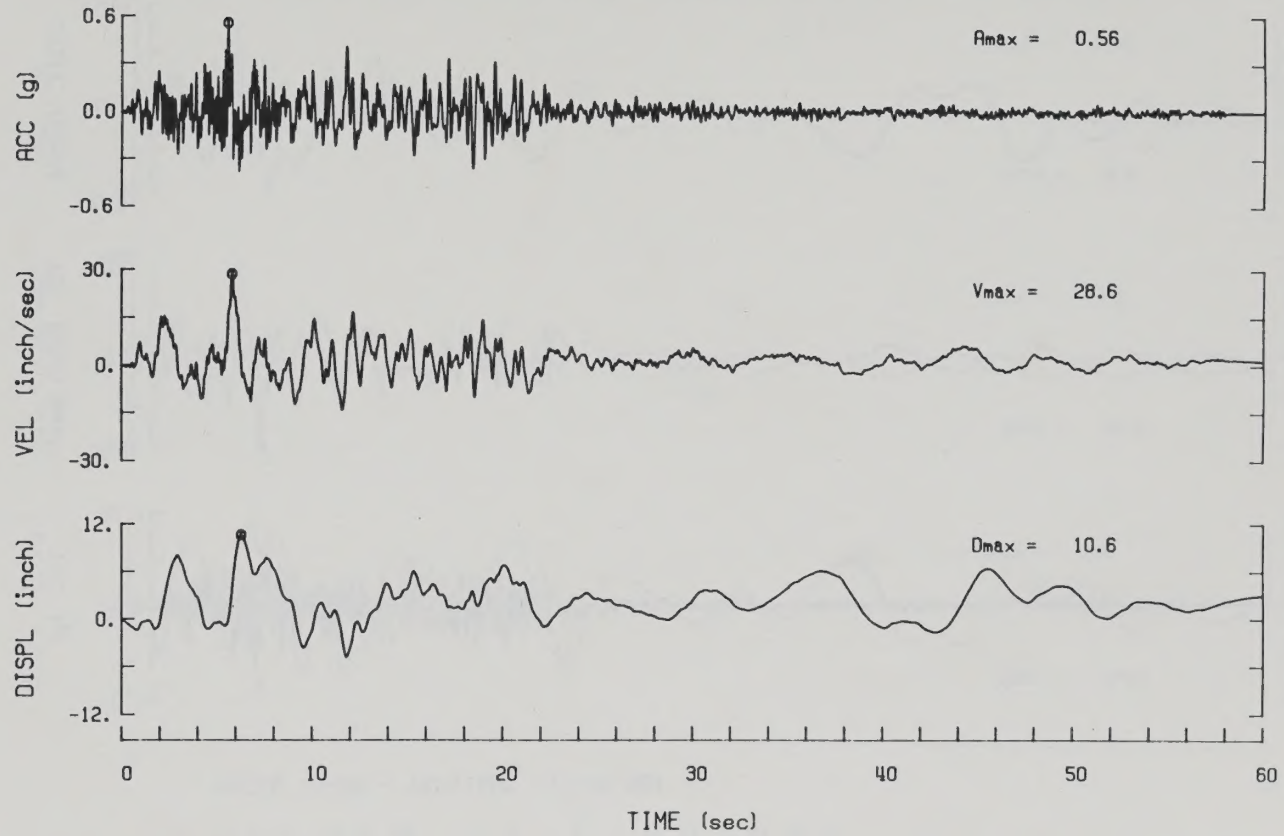
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MARIN ANCHORAGE - VERTICAL (TH1V-4B)



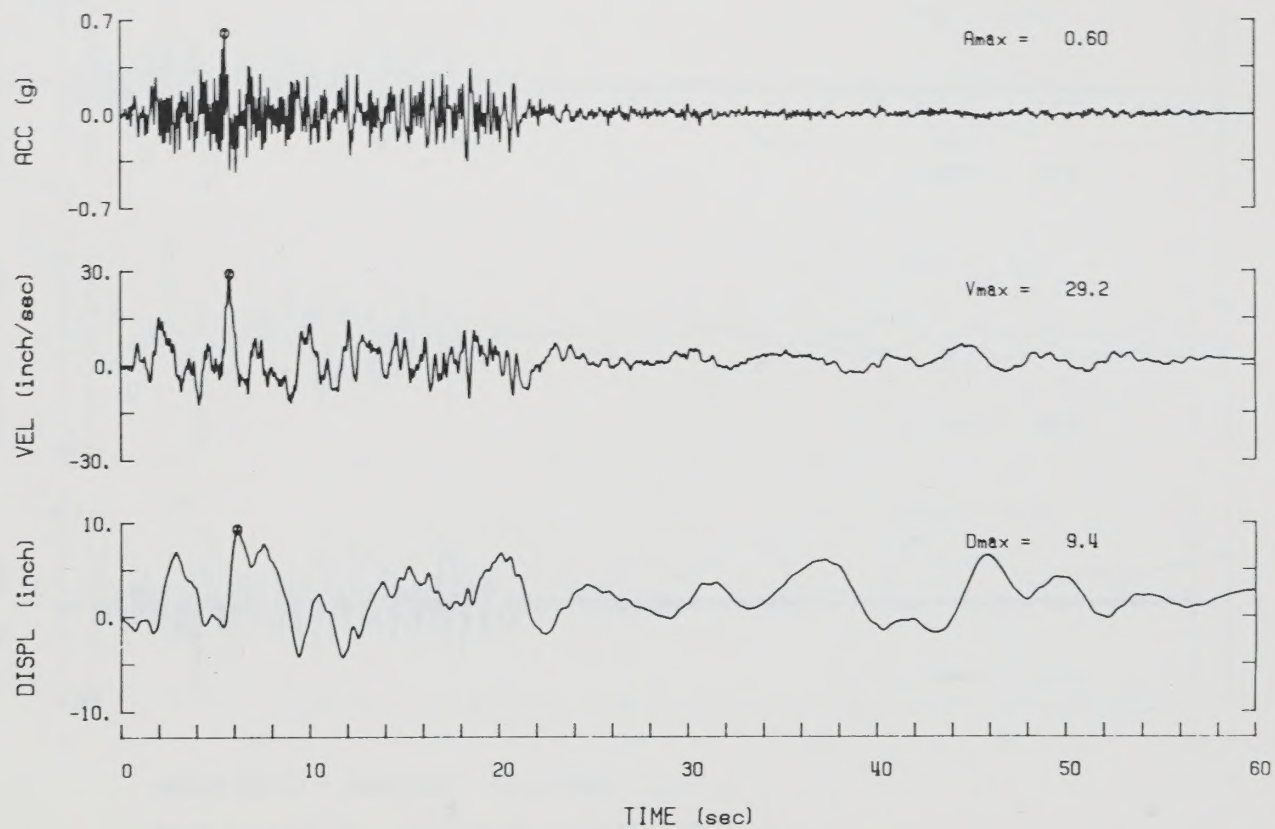
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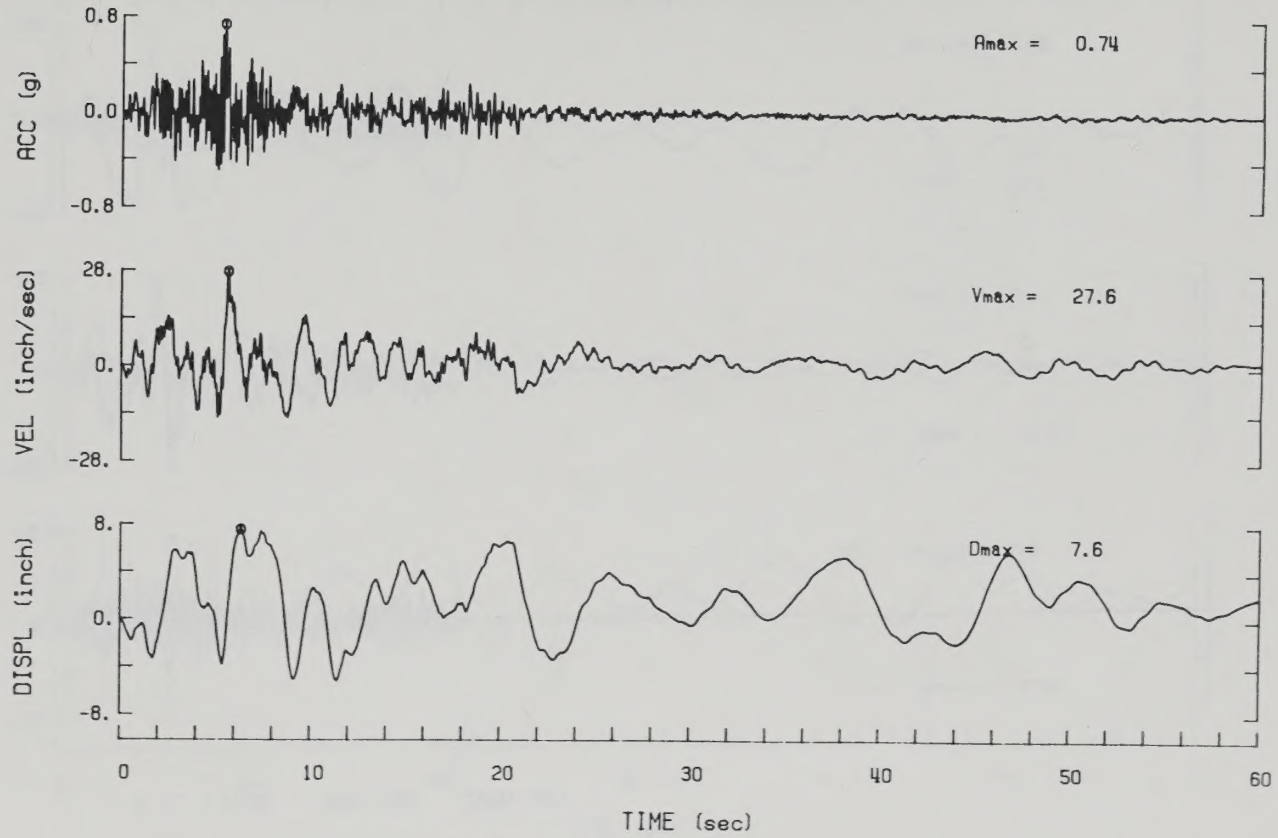
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MARIN TOWER - VERTICAL (TH3V-4B)



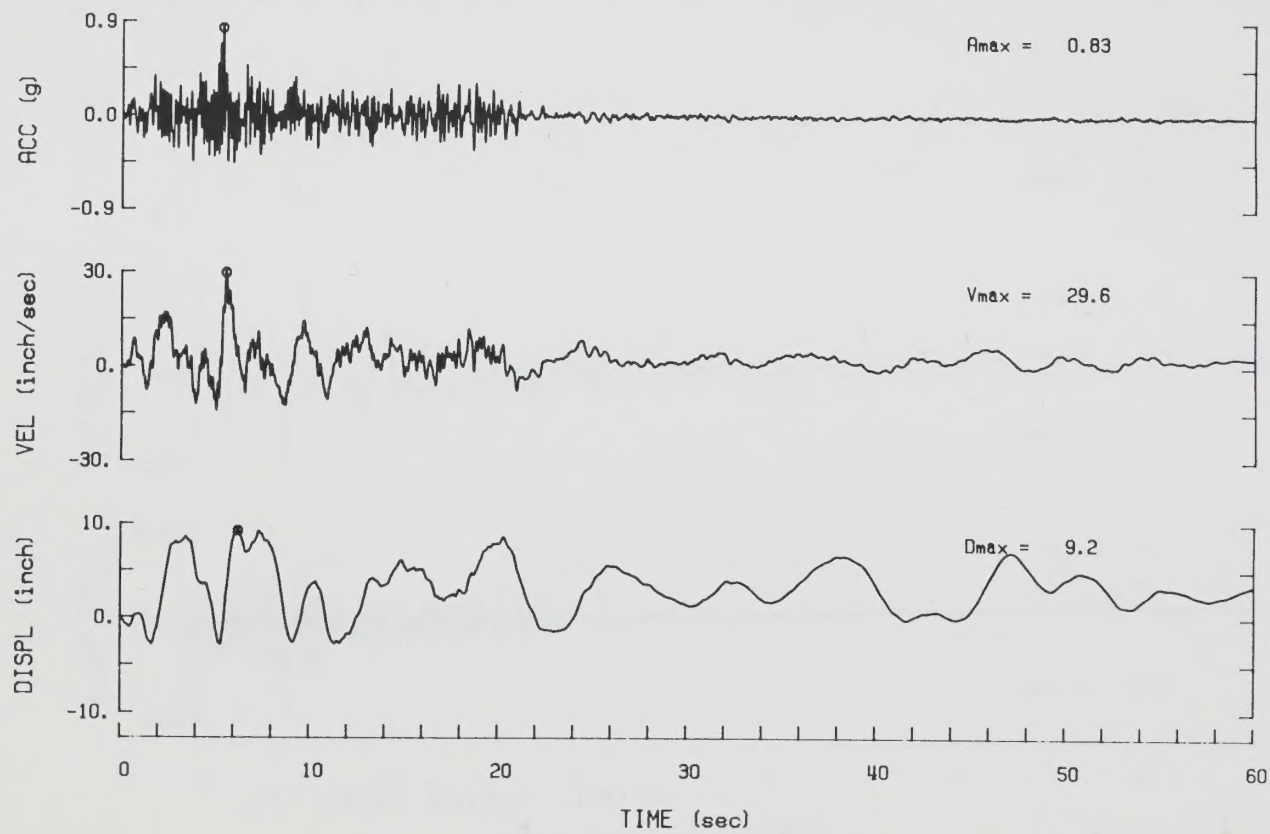
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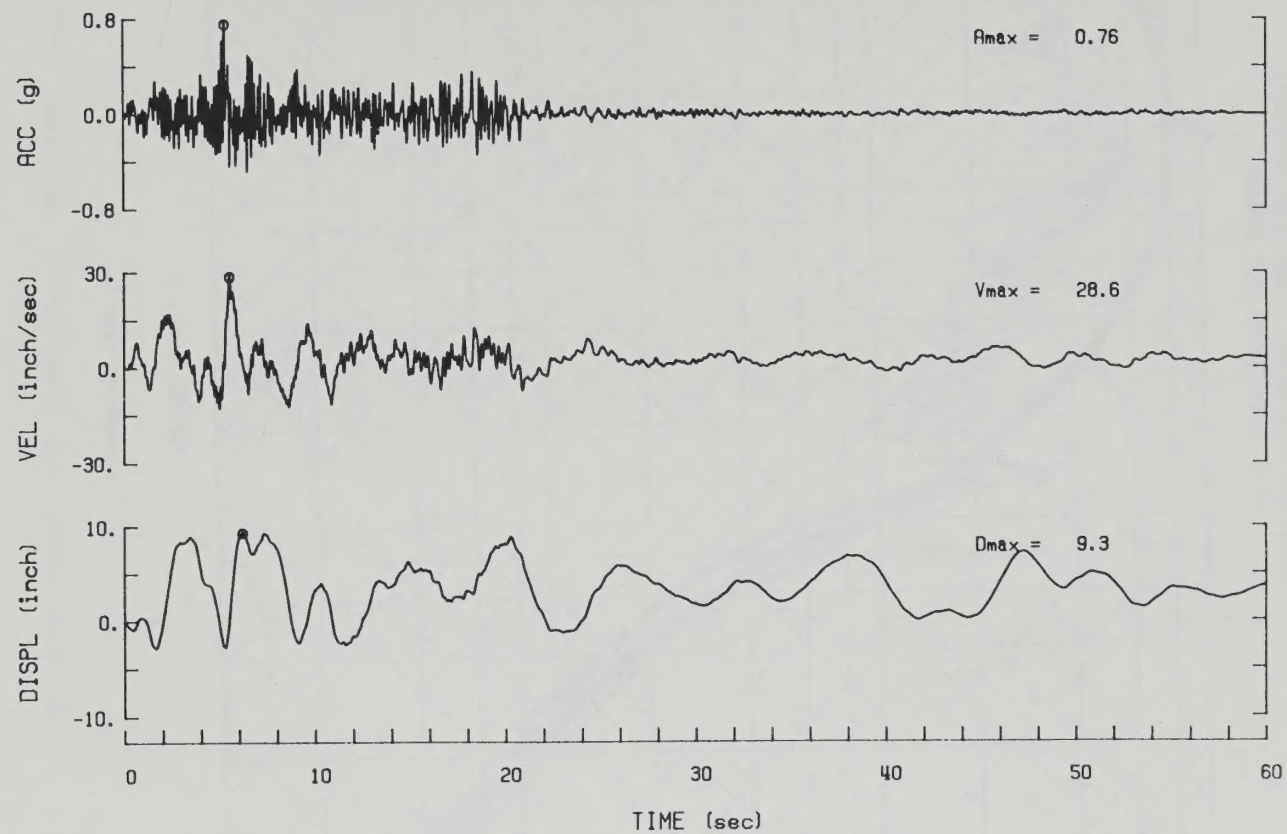
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S.F. PYLON - VERTICAL (TH5V-4B)



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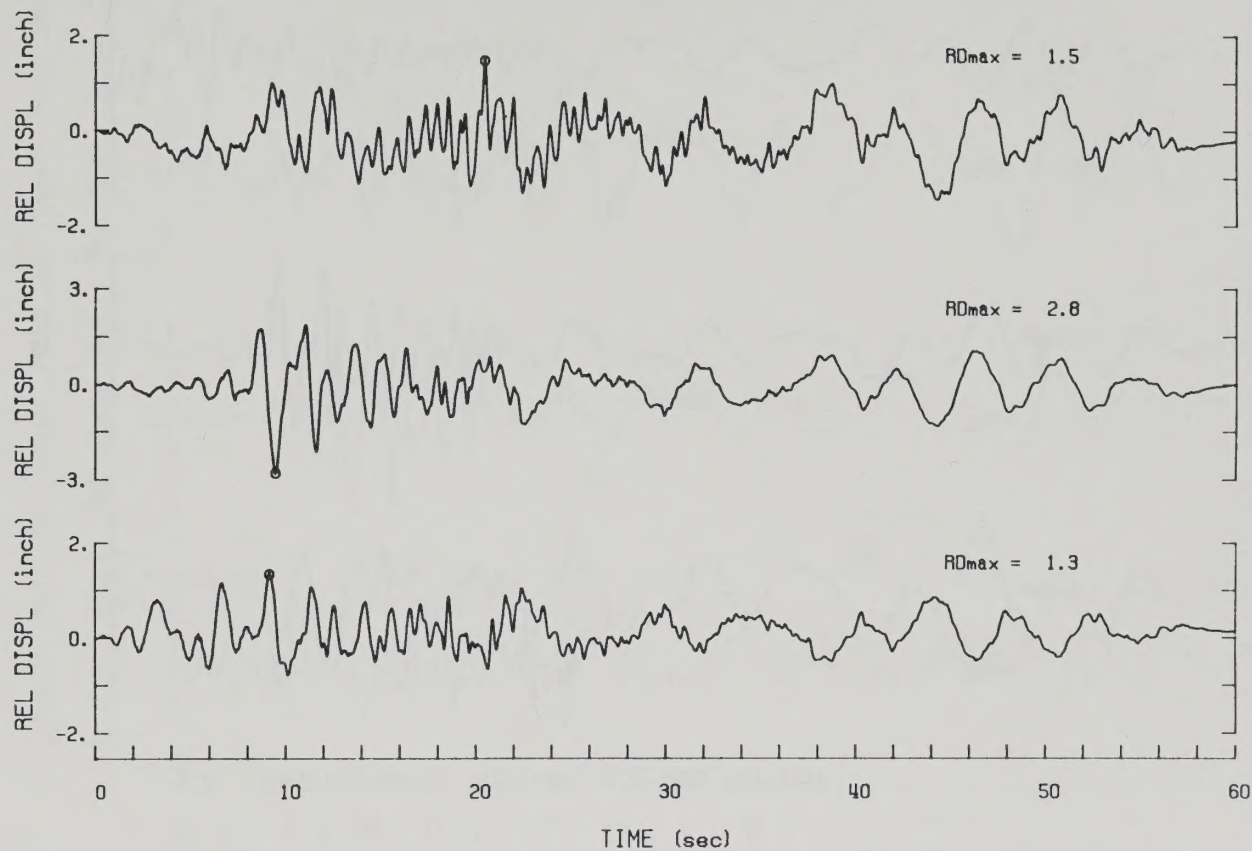


Vs = 4000 FEET PER SECOND



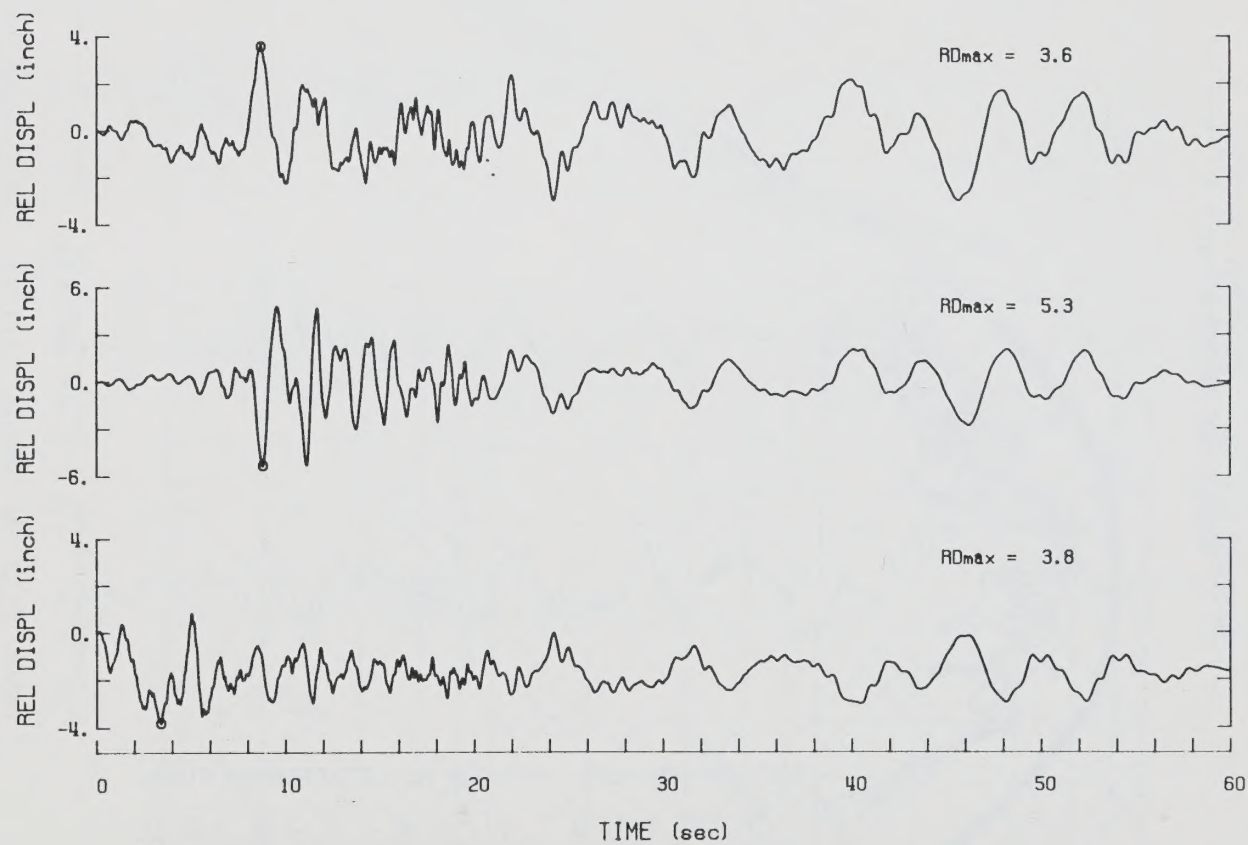
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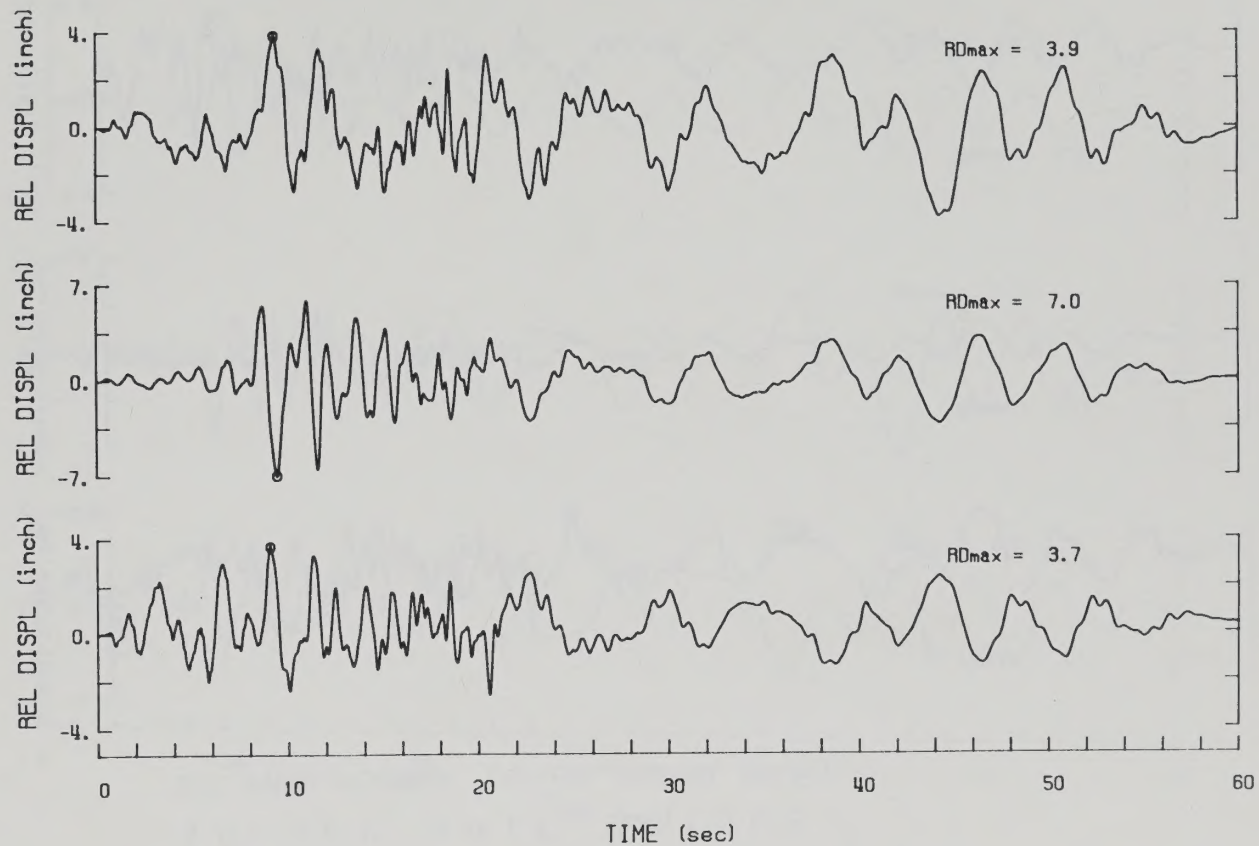


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S.F. TOWER/ANCHORAGE (TOP: LON, MID: TRN, BOT: VER)

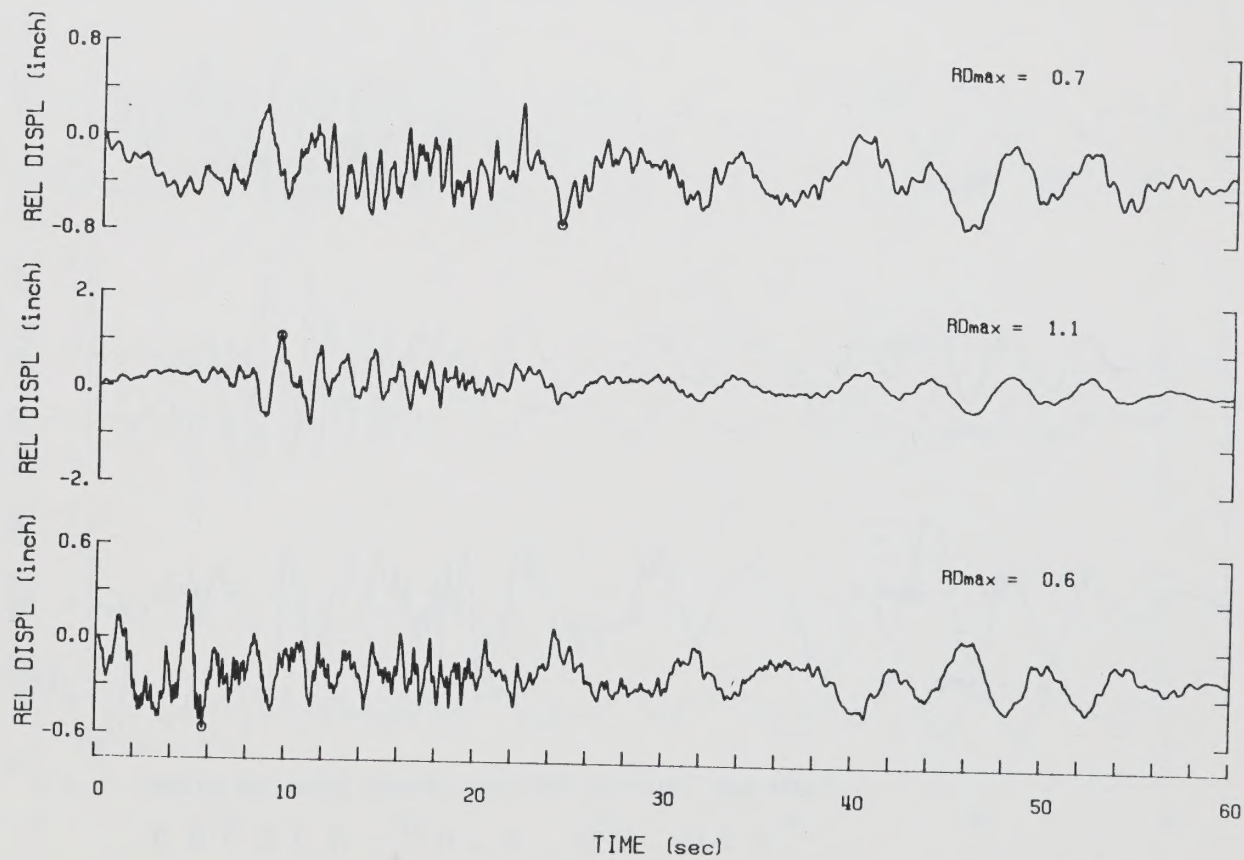


GOLDEN GATE BRIDGE
MARIN ANCHORAGE/TOWER (TOP: LON, MID: TAN, BOT: VER)



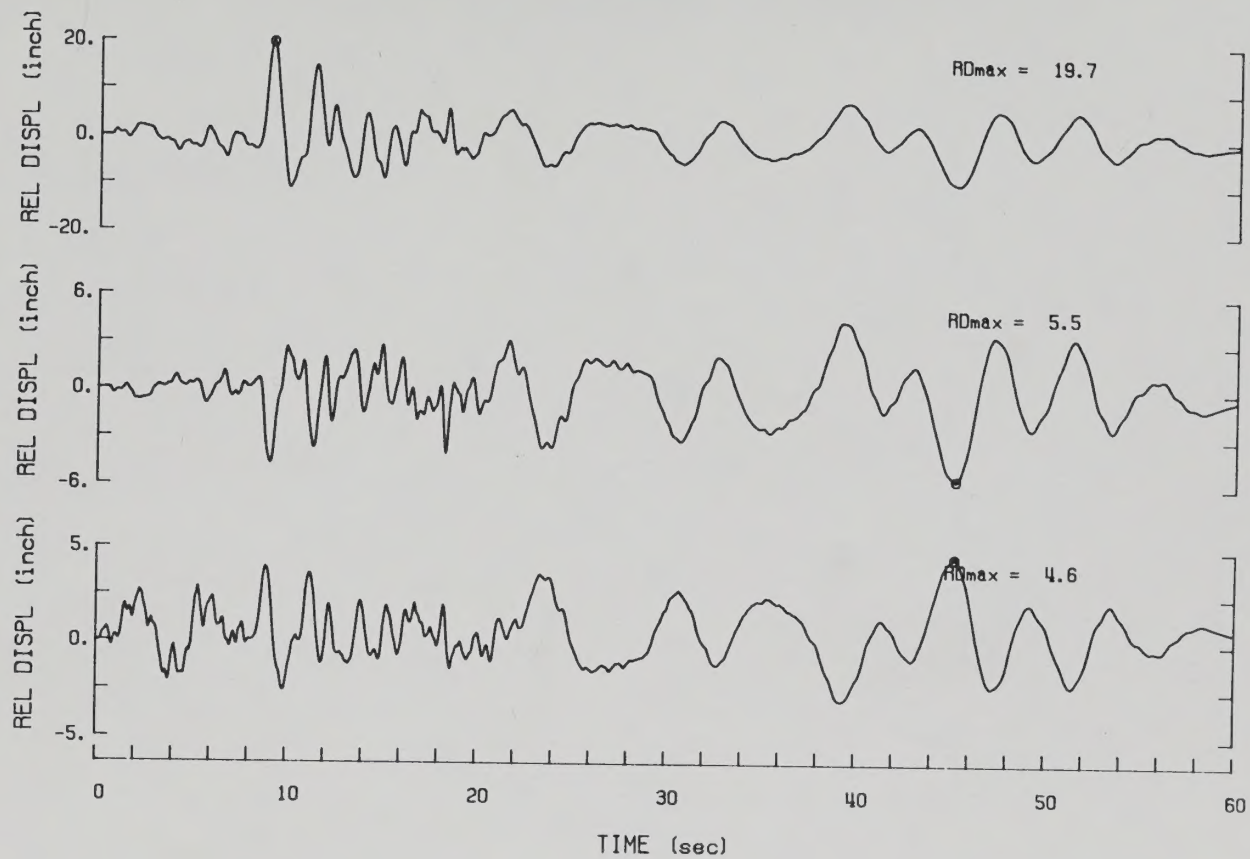
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S.F. PYLON/ANCHORAGE (TOP: LON, MID: TRN, BOT: VER)



GOLDEN GATE BRIDGE

MARIN TOWER/S.F. TOWER (TOP: LON, MID: TAN, BOT: VER)



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